ENGINEERING NEWS

A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. LII. No. 25. New York, December 22, 1904.

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CONCRETE-STEEL BRIDGE AT PLANO, ILL.; CHICAGO, BURLINGTON & QUINCY RY.

One of the numerous interesting concrete-steel railway bridges of recent construction is that built near Plano, Ill., to carry the Chicago, Burlington & Quincy Ry. over Big Rock Creek. It is a double track bridge, 44 ft. wide, with an arch of 75 ft. span, and having long concrete wing walls for the abutments, which bring the total length of the structure up to about 212 ft. It takes the place of a double-track steel bridge built in 1881, and having a 95-ft. deck truss main span on masonry piers, with two plate-girder approach spans of 24 ft. and 32 ft. on each side. The intermediate support for these spans consisted of a steel bent on concrete pedestals with cylinder pier foundations. This structure remained in service during the construction of the concrete bridge, as described below, and its position is indicated by heavy dotted lines in one of the accompanying drawings.

Fig. 1 is a view of the completed concrete bridge, and Fig. 2 shows the details of construction, with the position of the old steel structure indicated by the dotted lines, as noted above. The arch is of 75 ft. span, and is of the three-center type, having a radius on the intrados of 43 ft. for the main part and 20 ft. for the haunches. The radius on the extrados is 59 ft. The thickness at the crown is 3 ft., and the ends enclose the old masonry piers. It was at first intended to cut the face of each course of masonry of the old work in a series of steps whose top surfaces would be normal to the pressure line of the arch. It was finally decided, however, to cut out large and deep horizontal grooves or benches of triangular section in alternate courses, and in these to set corrugated bars 4 ft. long. The outer ends of these bars project into the concrete, making a bond between the concrete and the old masonry.

The arch is reinforced by Johnson corrugated steel bars, but the design is such that the concrete alone is of sufficient strength for the work, so that any strength due to the steel is in excess of the calculated requirements. The concrete arch is calculated for a load of 1,000 lbs. per sq. ft. The reinforcement consists of rows of ⅞-in. steel bars, 12 ins. apart, each row having four bars, 4 ins. and 6 ins. from the intrados and extrados, respectively. The bars of each pair are staggered to give a spacing of 6 ins., and between them run ¾-in. transverse bars about 2 ft. apart. The inner rows of curved bars at the extrados start from the old masonry pier, but at the sides these bars extend back to the foundations. Near each face of the arch are ⅞-in. radial rods which bond the arch to the spandrel walls. The construction is clearly shown in Fig. 2, and Fig. 3 gives the details of the arrangement of the concrete bars.

The abutment wing walls are about 55 ft. long, and are tied together by transverse concrete-steel beams 12 × 12 ins. in section. The steel work of each beam consists of a pair of angles, having their ends attached to shoes or anchors which are embedded in the walls. This is shown in Fig. 3. Between each spandrel wall and wing wall is an expansion joint, filled with ¾-in. of asbestos packing and covered on the top and side with a ⅛-in. lead plate, 10 ins. wide, having a groove which enters the joint. The upper surface of the arch is coated with ¼-in. of California asphalt. The quantities of materials are as follows:

Piles, white oak, 30 ft. long	100
Piles, pin oak, 24 ft. long	172
Concrete	3,350 cu. yds.
Broken stone	3,015 cu. yds.
Sand	1,508 cu. yds.
Portland cement	3,618 barrels.
Asbestos, pieces ¾ × 3 × 3 ins.	108
Asphalt	152 cu. ft.
Steel bars, ¾-in., 6,588 ft., 1.45 lbs.	9,952.60 lbs.
Steel bars, ⅞-in., 24,046 ft., 2.05 lbs.	49,294.30 lbs.
Total steel	59,246.90 lbs.

The arch centering, shown in Fig. 4, consists of ribs 3 ft. 6 ins. to 4 ft. 3 ins. c. to c., each rib consisting of three thicknesses of plank (one of 4-in. and two of 3-in.), and supported by posts and braces 6 × 10 ins., which are connected transversely by plank bracing, and longitudinally by 2 × 4 in. horizontal planks. The posts and braces rest on girts 12 × 14 ins., with caps, sills and posts 12 × 12 ins. The foundation for this centering consists of 20-ft. piles with 12 × 14 in. caps and bolsters, the latter carrying 12 × 12 in. sills directly under the sills of the centering. Between the two sets of sills are fitted the oak wedges, 10 ins. wide and 24 ins. long. The lagging is 5¼ × 2⅞ ins., dressed on the upper face and both sides. The forms are of planking, dressed on the inner face, and the high forms for the wing walls were supported from the outside by a system of X bracing transverse to the face of the wall.

The concrete of the arch (about 770 cu. yds.) is composed of 1 part Portland cement, 2 parts stone and 4 parts sand. For the foundations, spandrels, abutments, etc., a mixture of 1 cement, 3 sand and 6 stone and gravel was used. No special facing mixture was used, but the mortar was carefully laid against the form and all stones spaded back. There is a recessed panel on the face of the arch, and the corners are bevelled, while radial grooves extend across the haunches. The cement used was mainly of the Owl brand, but some of the Marquette brand was also used in the foundations.

The concrete was mixed in a machine of the open trough type, with a longitudinal horizontal shaft fitted with blades or paddles. This discharged the batch into four-wheeled steel hopper dump cars, with hopper doors operated by chains on a shaft across the top of the car. For building the arch, the concrete mixer was mounted on an elevated platform above the level of the track rails at the southwest corner of the old bridge, and in front of it was a transfer truck moving from the spout of the mixer laterally to a light track on a level with the main tracks. The dump cars were loaded from the spout of the mixer and traversed about 5 ft. to the track supported by trestling, on which they were run to the point where the concrete was to be deposited. The contents were then dumped through a chute and shoveled into place. As this work was done in cold weather, the water and sand were heated; no salt was used. To prevent freezing, a hose was connected to the boiler of the concrete mixer, and a steam jet kept playing on the concrete until it was certain that initial set had taken place. Observations with the ther-

FIG. 1. 75-FT. ARCH BRIDGE OF REINFORCED CONCRETE AT PLANO, ILL.; CHICAGO, BURLINGTON & QUINCY RY.

W. L. Breckenridge, Chief Engineer; C. H. Cartlidge, Bridge Engineer; G. H. Scribner, Jr., Contractor.

mometer were made at frequent intervals, and it was found that in no case had the temperature fallen below 32°F. within a period of 10 or 12 hours after the concrete had been deposited. Owing to the great width of the arch it was built in five longitudinal sections, each of which was a day's work, forming a monolithic arch. The five sections are held together by the transverse steel bars.

The material yard was close to the site of the bridge, with the stone pile close alongside the main track. From this yard two tracks for small

to rock (but of varying depth), is clay of different degrees of softness, with streaks of quicksand here and there. This bed of soft clay is full of water in the upper part (directly under the old hardened crust), but gets dryer and harder deeper down, and at the bottom there is much coarse gravel, with many glacial boulders. These conditions exist from the Chicago River south to 12th St., and from the lake shore west to the South Branch of the river. The elevation of the city was originally not more than 4 or 5 ft. above the average level of the lake, but after 1857 it

The last tall building which Mr. Burnha with stone pyramid foundations was the ance Exchange (now the Continental N Bank Building), erected in 1884. It was impossible to arrange economically the and other mechanical plant, because of th mids of masonry, and as the present R Rialto and Western Union buildings we being designed by the same firm, the ste grillage plan was developed into the I- grillage plan, which has since been used many tall office buildings everywhere i

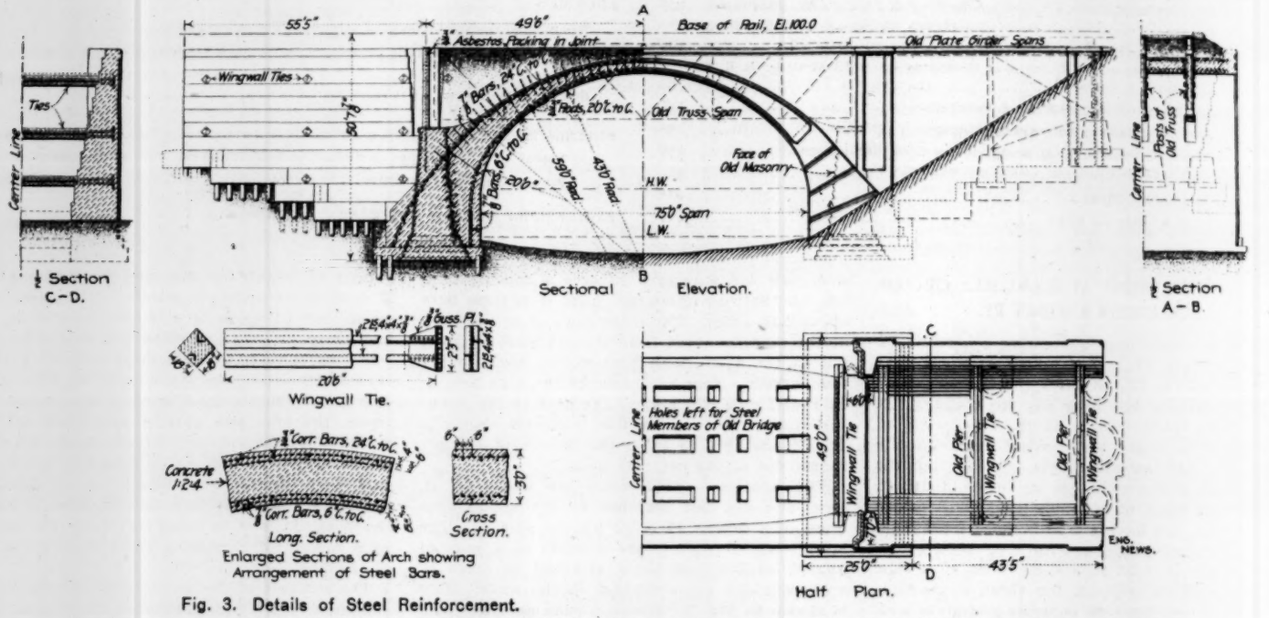


Fig. 3. Details of Steel Reinforcement.

FIG. 2. GENERAL PLAN, ELEVATION AND SECTIONS OF REINFORCED CONCRETE BRIDGE AT PLANO, ILL.

cars extended to the concrete mixer, the cars being hauled by cables up an incline to the measuring platform above the mixer. One track was for stone and the other for cement and sand.

As already noted, the concrete arch extended up into the steel trusses of the old bridge, these trusses being left in place to carry the traffic during the construction of the new bridge. To allow for this, holes were left in the concrete (as shown on the plan in Fig. 2), the steel members being cased in wooden boxing. These holes were smaller at the bottom than at the top, and after the removal of the steel trusses the holes were filled with concrete. The steel bars were stopped at these holes, but neither this nor the fact of the existence of the holes themselves affects the requisite strength of the arch. When the concrete of the arch was set sufficiently to stand loading, timber bents and blocking were erected upon it, resting on ledges formed for this purpose. This timber structure, which is shown in Fig. 4, supported the long ties of the floor system during the removal of the steel trusses and the placing of the permanent gravel filling.

The bridge was designed under the direction of Mr. W. L. Breckenridge, M. Am. Soc. C. E., Chief Engineer; and Mr. C. H. Cartlidge, M. Am. Soc. C. E., Bridge Engineer, of the Chicago, Burlington & Quincy Ry. The contract was let to Mr. Gilbert H. Scribner, Jr., Chicago. The filling, tracklaying, and removal of old bridge were done by the railway company. The work was commenced in August, 1903, and completed in June, 1904. The concreting of the arch was done between Dec. 1 and 31, 1903.

THE DEVELOPMENT OF SHALLOW AND DEEP FOUNDATIONS FOR CHICAGO BUILDINGS.

The development of foundation construction of large buildings in Chicago is a matter of rather exceptional engineering interest, particularly in view of the different methods employed to meet different conditions. The city is situated over a great clay bed, the upper part of which has hardened for a thickness of 9 or 10 ft. Below this, and extending about 90 ft. down

was raised by filling in the streets and alleys, the buildings being then jacked up to the new elevation, which was about 14 ft. above datum (mean lake level of 1847). For this work of raising buildings, the late George Pullman (inventor and builder of the Pullman car) was one of the principal contractors.

The contractors and builders of those days had determined that a safe load for buildings on the hardened clay crust was from 1¾ to 2 tons (net) per sq. ft. of foundation. The universal practice was to build piers of dimension stone masonry for foundations, the main or first floor of the building being above the street level and reached by steps, while in some cases there was a half-basement below and a first floor above the street level. Cast iron shoes on top of the piers supported the columns of the building, these columns being also of cast iron. As the buildings became higher and more important, the younger architects still followed the practice adopted by the earlier builders. But as buildings of eight and ten stories began to be designed, it was realized that the first floor must be level with the street, and that space between the street level and the level of the hardened crust must in some way be provided for the mechanical plant, including boilers, etc.

It was considered unsafe to excavate very deep into the hardened surface of the clay. Taking 2 ft. for the construction of the first floor left only 11 ft. to the bottom of the foundation. The result was that the basements were practically filled with stone pyramids, and there was no room for the boilers, engines and dynamos, which latter began to be used extensively about 1878. Neither was there room between piers for safety deposit vaults, which also then became a prominent feature of building construction. This question came up when the Montauk Block was designed in 1878 by Burnham & Root, a young firm of architects composed of Mr. D. H. Burnham and the late Mr. John Root. The ordinary stone pier construction was used for part of the foundations, but in order to obtain room for the boilers they devised a grillage of steel rails embedded in concrete, forming a shallow spread foundation.

United States. Mr. Wm. Sooy Smith, M. Am. Soc. C. E., was consulting engineer in regard to this use of steel in concrete.

It may be noted here that the important step of lowering the first floor to the street level resulted in some cases in a rental for that floor of over twice what had been received for the rental of the former elevated first floor and half basement. In regard to the Montauk Block, also, an interesting feature was the support of the party wall of a tall building adjacent to it. Mr. Peter B. Wight, now Secretary of the Illinois State Board of Examiners of Architects, was consulted by the architects, and suggested the use of jack screws in the foundations, so that the party wall could be leveled up as required. This plan was adopted with success. The Montauk Building was pulled down in 1903 to make room for the new First National Bank Building.

The principle of the "floating" foundation was to carry the shallow spread foundations on the thin, hard stratum covering the deep bed of soft clay underlying the city, keeping the load per square foot at a safe limit and avoiding all disturbance and penetration of the clay crust. It consisted in building a concrete-steel grillage or platform for each column, this footing having rows of I-beams embedded in concrete. This shallow footing and the steel columns replaced the heavy concrete and stone footing and piers which had previously been used, and it not only reduced the load on the foundation, but gave ample basement area (otherwise obstructed by masses of masonry), which was a matter of growing importance. The area of each footing was determined by the load on the column which it supported, and the load per square foot which it was considered safe to impose upon the soil. The practice of thus calculating the loads on individual columns and proportioning the area of foundation to carry the load thus determined, was first adopted by Mr. Wight and his contemporaries, just before Burnham & Root began the practice of architecture.

While the "floating" foundation was used almost universally for several years, at the present time the deep foundation system is being

almost exclusively, for two principal rea-

The increased value of building ground, leads to a demand for sub-basement space ich the power plant and other appur- es may be placed, leaving the basement y free for business purposes. It will be understood also that the extensive and mechanical plant of the modern building res very much more height than that of the buildings, quite out of proportion to the rence in size of the buildings.

The disturbance of the clay bed has led to some irregular settlement of older buildings having the "spread" or surface foundations. This movement has not caused, and probably never will cause, any danger to the structures, the steel cage construction being sufficiently flexible to take up the distortion, but careful levels taken periodically prove the unequal settlement beyond dispute. In one particular case deep foundations have been sunk to stop the settlement of the original spread foundations, which was becoming serious.

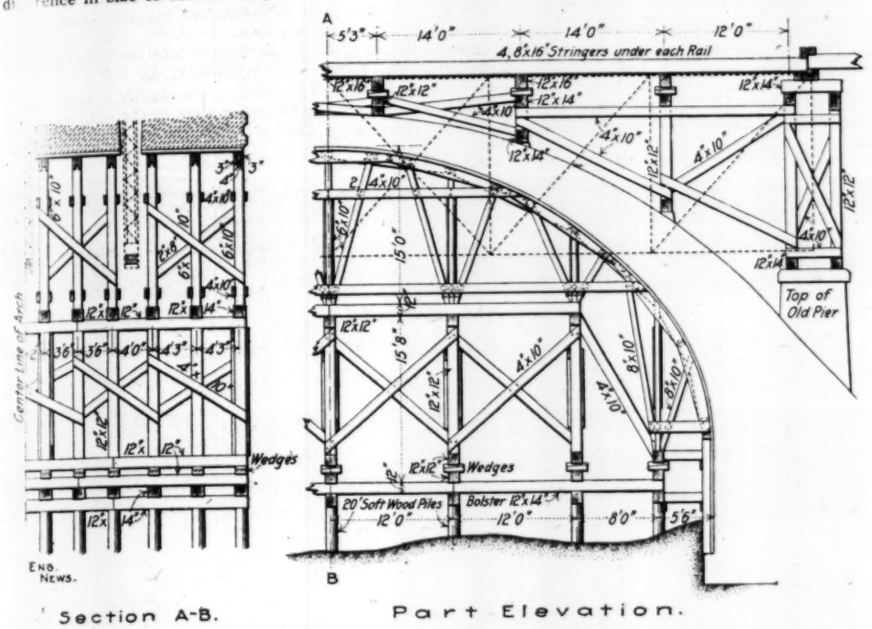


FIG. 4. DETAILS OF CENTERING FOR CONCRETE ARCH AT PLANO, ILL.

Second: The desire for greater stability. While the "floated" foundations were sufficient to support buildings of 8 or 10 stories, yet with the heavier loads of taller buildings the crust itself would deflect. The load per square foot of foundations would be no greater (due to the enlarged area of each footing), in fact it might be even less; but the total load concentrated on a lot for a 16-story building being nearly double that of an eight-story building for the same area, it was found that the architect had to deal with not only the pressure of 1¾ net tons per sq. ft. of foundation work, but also a double aggregate load on the crust beneath the building, the area of the crust under load remaining the same.

The opinion is held by some architects (but disputed by others) that the construction of deep foundations as well as the construction of water tunnels, telephone tunnels, etc., has rendered the "floating" type of foundation undesirable for the larger classes of buildings in the future, owing to the disturbance of the clay. This clay, as already explained, is soft and wet in places and dry and hard in others; but the soft upper clay (directly underlying the hardened surface crust) is of a doughy or plastic nature, so that when any cavity is formed in it, the clay will gradually close it up, the movement of the mass extending for considerable distance.

Thus in sinking shafts or driving tunnels, if any space is left outside the lagging, the clay body will move until this has been filled in. Shafts left unlined in the soft clay will gradually close, and in one case where it was attempted to carry on pile driving at the same time as the shaft sinking, the upper part of the shafts (above the working chambers, which had reached hard clay) began to close up. This plasticity of the clay has been shown by numerous experiences in pile driving and foundation work, as noted in articles in our issues of Sept, 21, 1893, and Jan. 27, 1898. Driving piles in one place would displace piles at other places or cause them to rise. After the construction of the new Tribune Building (with deep concrete pier foundations) in 1903, the Hartford Building, on the opposite side of the street, was actually ¼-in. higher than before.

Pile foundations have been used in a number of cases, sometimes relying to some extent upon the skin friction of the piling, but generally having piles driven to the underlying hardpan or rock. The site of the Chicago Stock Exchange Building, built in 1893, was found to consist of a 55-ft. stratum of clay which contained 55 lbs. of water per cu. ft., and seemed likely to settle indefinitely under load. Gen. Wm. Sooy Smith, M. Am. Soc. C. E., the consulting engineer for the work, concluded to go down to the hard clay beneath, using piles mainly. Near adjacent buildings, however, where pile driving might be dangerous to the existing structures, 5-ft. shafts were sunk to a depth of 55 ft., being lined with oak poling boards 4 ft. long and braced inside by heavy iron hoops. When the hard material was reached, the shaft was widened out in conical form to a bottom diameter of 7 ft. 6 ins. These shafts were then filled with concrete, forming foundation piers.

Pile foundations were also adopted in 1893 for the Chicago Public Library. The piles averaged 54 ft. long and were driven 52½ ft. by a Vulcan-Nasmyth steam hammer (27 ft. in soft clay and 23 ft. compact clay). It was required that they should not carry over 30 tons per pile, and tests made showed no settlement under 50-ton test loads maintained for 11 days. The Fisher Building has a combination of pile and spread foundations. Under each column was driven a group of 25-ft. piles to consolidate the wet clay. A 6-in. bed of concrete was packed around the heads of the piles, and this was covered with a grillage of steel beams embedded in concrete. For the new post-office it was at first intended to have concrete piers. For this purpose steel cylinders, 12 to 15 ft. diameter, were to be sunk to rock at a depth of 72 ft., and then filled with concrete. The estimated cost was $200,000. Gen. Wm. Sooy Smith, who was consulting engineer for the foundation work, made an investigation, and decided to use piles. The hardpan lies at 72 ft. below the street grade, and was reached by 50-ft. piles driven in the bottom of excavations 28 ft. deep.

As far back as 1892, Gen. Sooy Smith had strongly advocated carrying foundations to rock (at depths of 60 to 100 ft. in places), or to the hardpan overlying it. He proposed sinking wells or shafts to a sufficient depth below datum, and then driving piles to the solid foundation. When the rock was comparatively near the surface, the wells might be sunk to it, and in either case the wells would be filled with rubble masonry or concrete. It is this well system of foundations which is now being used almost exclusively, but without any piling, the wells or shafts being sunk to reach the stable material (either rock or hard clay), and then chambered out to form enlarged footings. The spread foundations are still occasionally used, but they are now quite exceptional.

In all cases, the shafts are sunk as open wells, being lined with vertical tongued and grooved planking, braced by iron hoops. These hoops are made in semicircular form, with the ends bent inwards to form flanges. A great variety of material is met with, and there is always uncertainty as to what materials and difficulties will be encountered at any particular site. Test borings do not give thoroughly reliable results, and only the actual excavation discloses the conditions. Clay of various degrees of wetness, quicksand pockets, nests of boulders, etc., are liable to be struck. In no case has the pneumatic process been employed, but in one case steel cylinders had to be substituted for the timbering, in order to enable the shafts to be sunk through a pocket of quicksand.

The use of steel cylinders, instead of wooden lining, as above mentioned, was required for the deep foundations of the Chicago Edison Co.'s building at 84 Market St., in order to carry the shafts through a bed of quicksand. The architects were Shepley, Rutan & Coolidge, of Chicago, and for particulars of the foundation work we are indebted to the contractors, the Wells Brothers Co., also of Chicago. The area of the site was about 75 x 90 ft., and the quicksand covered the entire area, being 10 to 12 ft. thick, with its surface 100 ft. below the street grade or 85 ft. below city datum. No clay or hardpan was found below the quicksand, but there was a layer of boulders varying from the size of cobblestones to stones 4 and 5 ft. diameter, overlying the bed rock. There were 24 cylinders in all, 18 being 6 ft. 6 ins. and six being 8 ft. 6 ins. diameter. The ordinary method of sinking the shaft with wooden lagging and iron rings was followed until the quicksand was reached. A steel cylinder was then put down, made in three sections with vertical joints fitted with angle iron for the connecting bolts. The excavation was at once started, and as the sand was removed the cylinders settled very easily by their own weight until they rested on the boulders overlying the rock. The boulders had then to be drilled and split. It was found hard to settle the cylinders after they had stood for some days, and it was necessary to use jacks to force them down so that the bed rock could be cleaned and the concrete filling put in.

One advantage in the use of the system of wells is that after their construction a subcellar or sub-basement can be excavated without affecting the foundation, and this was done in the interior area of the new store built for Schlesinger & Mayer, but which is now owned by Carson, Pirie, Scott & Co. While this does not affect the foundations of the building itself, it may affect adjacent buildings unless great care is taken to hold back the clay, the plastic and flowing character of which has already been referred to. This trouble was encountered at the new Tribune Building, where the central part of the site was excavated first, resulting in a movement which caused some trouble before it could be controlled. In preparing for the new 17-story Strong Building at State and Adams Sts., the architects, Holabird & Roche, took steps to prevent trouble of this kind, by first excavating a trench around the site and putting in a heavy steel and concrete curb-retaining wall, strong enough to resist the outside pressure. This trench was lined with steel sheet piling, and heavily braced. Mr. Peter B. Wight, of Chicago, has suggested that the exterior wells may be put down at a distance apart equal to the

diameter of the wells, the intermediate spaces being then filled in by other wells of circular or rectangular section, so as to form a continuous wall.

Recent examples of buildings with deep foundations are as follows: Tribune Building and Strong Building, 17 stories each; Holabird & Roche, architects. First National Bank, Merchants' Loan and Trust, Railway Exchange and Marshall Field retail store; D. H. Burnham & Co., architects. Hartford Building (new portion); H. I. Cobb, architect; Schlesinger & Mayer retail store, 12 stories, Louis H. Sullivan, architect. The McNeill Building (Holabird & Roche) has caisson foundations under the party walls, and pile foundations for the interior piers. The same plan was adopted by Frost & Granger for the Hibbard, Bartlett & Spencer Building, and the La Salle railway station. Under the First National Bank Building the piers are 6 ft. to 10 ft. diameter, and go down to rock, which is found at a depth of 102 ft. below the street grade. At the Marshall Field store and at the Railway Exchange the piers extend to a depth of about 90 ft. below the street grade, where they rest on a very hard clay or hardpan, which overlies the rock to a thickness of about 30 to 40 ft. In these cases the piers were enlarged at the bottom, as already described, so as to reduce the pressure to about 5 tons per sq. ft. of surface covered. At the Tribune Building, the wells struck hard material at various depths.

It is worth noting that in several cases the foundations for new buildings have been put down under old buildings without interfering with the use of the latter, except that the basement is vacated for the use of the contractor. In this way the period during which the site is unproductive of rent is reduced to a minimum, and when this work is done it is a comparatively quick and easy matter to pull down the old structure and erect the new one.

Where the site is cleared before the construction of the new foundations, the surface excavation is done by hand. This usually involves much labor and hard treatment of the teams which pull the loaded wagons up inclined roadways out of the pit, although extra teams and even a hoisting cable and engine are sometimes employed to help the wagons. It seems to be generally assumed that the teams must go into the pit, but at two of the new Chicago buildings two different methods have been employed to do away with the trouble and delay (to say nothing of the cruelty) of this practice. At the Railway Exchange the clay was loaded into small end-dump cars, which were hauled up by a cable and winding engine to a platform projecting over the street, and high enough for wagons to drive under it. On this platform the car was automatically tripped, and dumped its contents through a trap into the wagon standing in the street. At the First National Bank, the teams took the wagons to the edge of the pit, and were unhitched; slings were then put around the wheel hubs, and the wagon was picked up by a derrick and lowered at any desired part of the excavation. The loaded wagons were hoisted out in the same way. In part of the work for this latter building the hoists for the shafts were set above the level of the first floor, and the clay was dumped into a hopper supported on a timber framing high enough to allow a wagon to be driven underneath. This hopper had a movable bottom, which was closed by a chain and shaft operated by a hand wheel. The advantage of this arrangement was that the digging could go on when there were no wagons to receive the material, and when the wagons came they could be loaded quickly, by opening the bottom of the hopper. The network of tunnels built for the automatic telephone service of the Illinois Tunnel Co., but now used also for freight transportation purposes, may be utilized in future for the handling of material excavated in foundation work. The proposition is to sink a shaft at the site of the work, and build a lateral tunnel to the nearest line of the tunnels, which are at a depth of about 26 ft. The material would be lowered down the shaft into dump cars, which would be hauled by electric locomotives to a dumping place either at the river (into scows) or at the lake front for filling in the new park site. The company has, in fact, recently built this dumping point on the lake front, approached by an open cut across the present park, and a bridge over the Illinois Central Ry. The sides of this cut are supported by steel sheet piling of the Jackson pattern.

A considerable amount of this open-caisson deep foundation work has been done, and more is in progress. The diameter of the caissons is dependent upon the load which they are to carry. The smallest are 3 ft. 2 ins. in diameter, and the largest about 8 ft. For some time architects generally specified that they were to be sunk to satisfactory hard-pan, which in the down-town section of Chicago is found at a depth of from 70 to 90 ft. below city datum. Of late, however, architects have, as a rule, been specifying that the caissons must go to rock, which in the same location is found from 90 to 110 ft. below datum. Where the caissons are carried to hard-pan, they are belled out to twice the diameter of the caisson proper, the belling being generally done at an angle of 45°. The wells are dug by hand to the exact size required, plus the thickness of the lagging, and sunk to a depth of 4 ft. Then the lagging is put in, with generally three iron rings, which go, of course, on the inside. Then, another 4 ft. depth of well is dug out, and another ring of lagging put in. The dirt is removed in a bucket, operated by a windlass. Sometimes the windlass is worked by hand or by an individual steam hoisting engine, but where there are a number of caissons it is quite customary to have over the head of each caisson a spool on a shaft having a wheel driven by a continuous running rope operated by a steam engine or electric motor. One rope will serve a row of shafts; it runs constantly, and [illegible] at each shaft coils the bucket rope aro[illegible] spool when the load is to be hoisted. T[illegible] ging is generally pine, 4 ft. long, from 2 [illegible] thick, and not over 6 ins. wide. In sinki[illegible] caissons, the most important point to b[illegible] for is not to dig the hole larger than [illegible] lutely necessary. If there are any voids [illegible] the lagging, settlement of adjoining pre[illegible] almost sure to occur, as already noted.

On all large operations it is customary [illegible] mechanical concrete mixing machine. Ar[illegible] differ in their specifications regarding the [illegible] iting of the concrete. In many cases the[illegible] the concrete to be thrown into the hole. [illegible] tain cases they require it to be lowe[illegible] buckets; and in other cases require a chut[illegible] used. In all cases they require the concret[illegible] thoroughly tamped. The ordinary specifi[illegible]

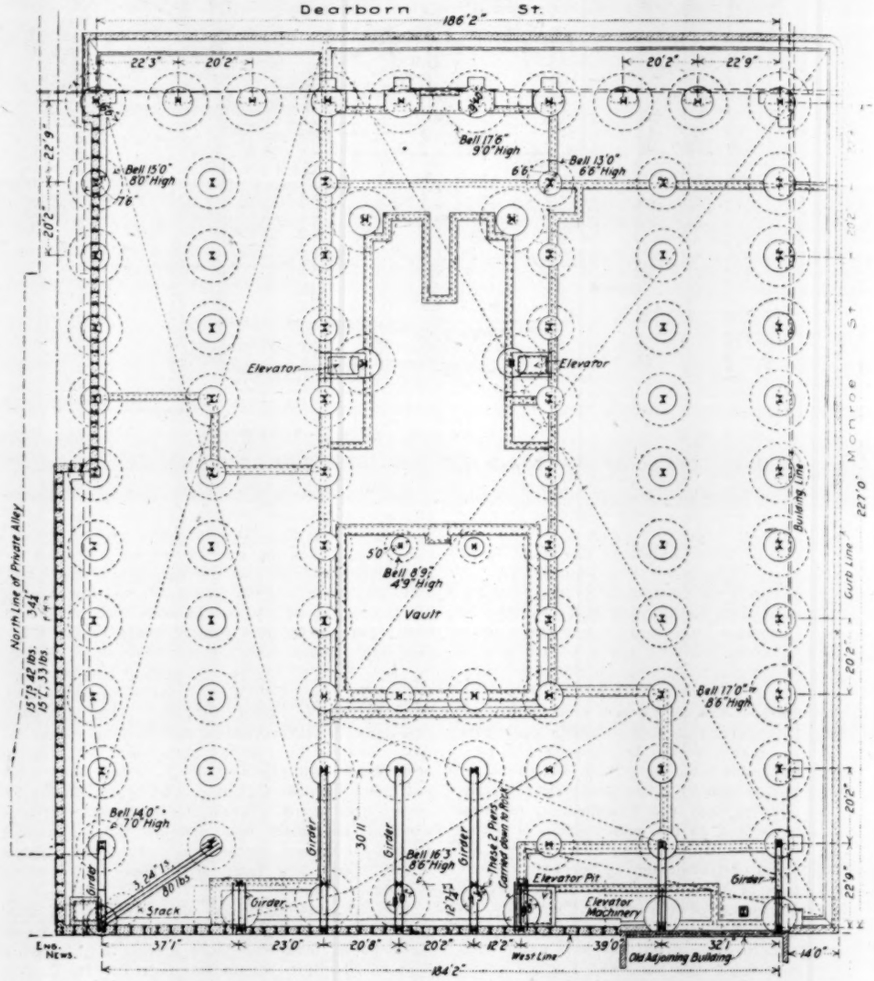


FIG. 1. PLAN OF FOUNDATION PIERS OF FIRST NATIONAL BANK BUILDING, CHICAGO.
D. H. Burnham & Co., Architects.

for the composition of the concrete is as follows: 1 part of high-grade Portland cement, 2 parts clean, sharp sand, and 4 parts broken stone. Under ordinary conditions a caisson or shaft can be completed in from 10 to 12 days, day and night work. Some engineers load these caissons up to as high as 50,000 lbs. per sq. ft., but the general practice is to load them not to exceed 40,000 lbs., and a few engineers argue that they ought not to be loaded over 35,000 lbs.

The arrangement and dimensions of foundation wells for the First National Bank Building are shown in Fig. 1, and Fig. 2 shows the footing chambers and the steel capping. Fig. 3 shows the heavy concrete curb retaining wall which carries the sidewalk and forms the space for the basement and sub-basement. Fig. 4 is the detail of the retaining wall on the west side, which is reinforced by 15-in. 42-lb. vertical I-beams, 3 ft. [illegible]

...c. to c., resting on a sill consisting of a 15-in. ... steel channel. This is for a depth of 26 ft. ... On the north side is a similar wall with ... The sill is, of course, embedded in the ...

... to the foundation work of the Schlesinger ... ayer store building, already mentioned, we ... the following from a paper read in Febru-

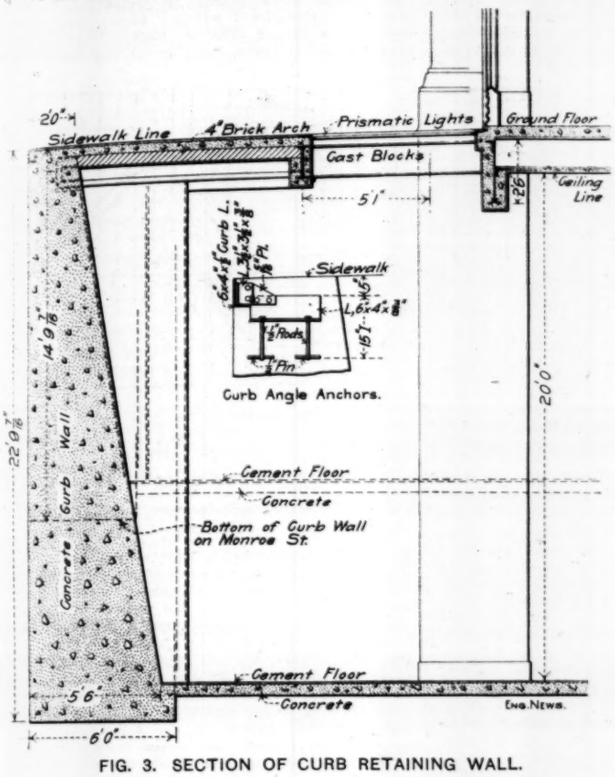


FIG. 3. SECTION OF CURB RETAINING WALL.

We put in 56 wells, each 75 ft. deep, and although we used the heaviest kind of shoring to hold the banks in place, as a matter of fact it was not necessary to put in any. The banks will retain themselves. At the bottom of these wells, when we came to widen them out for the bell, the clay was so heavy that it weighed 150 lbs. to the foot. At a depth of 75 ft. we came to shale that was almost chip dry and so hard that the men could only chip off about 2 ins. at a blow. I do not know how far it is to bed rock, for although I had borings made at the bottom of the wells, the shale got so hard 12 ft. down that the men could not work the auger. I was satisfied to build on that shale for a foundation.

It may be that in places the clay will be cracked, in which case water will come into the wells to a certain extent, but once you get down to the shale, which lies about 75 or 80 ft. below sidewalk level, you will have a

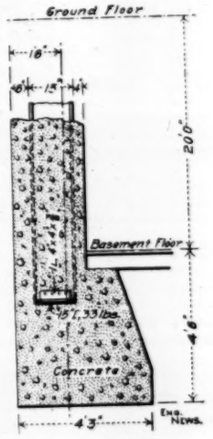


Fig. 4. Retaining Wall.

ary before the Illinois Chapter of the American Institute of Architects, by Mr. Louis Sullivan, architect for the building. As already stated, however, the conditions described by Mr. Sullivan are by no means universally encountered, there being great uncertainty as to what materials and conditions will be found, and at what depth a stable foundation will be found, at any given site. In the discussion of this paper Mr. Frederick Baumann stated that he had found the soft clay extending as deep as 30 ft. below the street grade, but Mr. Sullivan thought that by going down to 40 ft. the stiff blue clay would have been found. He thought that although the soil is full of mud holes and soft spots of varying depth, underneath all is a good hard clay and shale foundation, dry and non-compressible.

We have been used to believing that there was a deep substratum of pulp soil and that we could put our basements down 10, 12 or 15 ft., but no deeper. I have had borings made and I find that they are fallacious, and that the earth brought up does not represent the true character of the soil. At the Schlesinger & Mayer building the borings were most unfavorable, but we started to sink the wells. Instead of soft clay, we found, after having gone about 15 ft. below sidewalk level, a blue clay. This is not the kind of clay that we have commonly called blue clay, but it is a stiff clay, that kept getting stiffer and stiffer as we went down. At the depth of 50 ft. this clay was so stiff that the men were unable to dislodge a shovelful without dipping their shovels in water three or four times. Instead of finding water, as we had expected, we had to send water down to the men in buckets.

You may know what a task it was to excavate this clay when I tell you that it took two months to dig out a space 50 × 140 ft. to a depth of 26 ft. I believe that there has been a complete misapprehension as to the nature of the soil under the city. We used to go down through a lot of muck and rubbish in our excavating and then at a depth of about 12 or 13 ft. we would strike what was supposed to be blue clay, which would be from 2 to 4 ft. deep. As a matter of fact that is merely an incident, and if we will only go far enough we will find the finest kind of soil.

safe foundation to build upon. Therefore building subbasements is a simple proposition and does not require scientific engineering, but only hard labor. That shale is the original glacial drift. It is hard, and you can moisten it and there will be no smell of clay about it. We found in it fragments of block stone, and, as I have said, it is so hard that it could only be dislodged by a strong man at the rate of about 2 ins. a blow. There is absolutely no chance for a building to settle if it is built on caissons, resting upon this shale. Pile foundations are not safe, but caisson foundations are absolutely safe, if sunk deep enough.

We have at various times published a number of articles relative to the building foundation problem in Chicago and descriptive of different examples of foundation work. The following list includes those of most importance:

Steel Spread Foundations, Aug. 8, 1891; The Chicago Foundation Question, Oct. 13, 1892; Chicago Buildings and Foundations, Oct. 13, 1892; Pile Foundations for Chicago Public Library, July 6, 1893; Foundations of the Chicago Stock Exchange Building, Aug. 31, 1893; Pile Foundations in Chicago, Sept. 21, 1893; Foundations of Old Colony Building, Chicago, Dec. 21, 1893; Column Support Adjustable for Settlement, Oct. 17, 1895; Foundations of Fisher Building, Chicago, Oct. 17, 1895; Pile Foundations of the Chicago Post Office, Jan. 27, 1898; Foundations of the La Salle St. Station, Chicago, Aug 6, 1903.

RECENT PRACTICE IN LAUNCHING WARSHIPS IN THE UNITED STATES.*

RECENT LAUNCHING PRACTICE OF THE ATLANTIC COAST.—By R. H. M. Robinson,† M. Soc. N. A. & M. E.

In considering the question of the launching of the U. S. S. "Connecticut" at the New York Navy Yard, Sept. 29 of this year, it became desirable to obtain data as to the best practice in launching similar ships at private yards, as there was no established practice at this yard for launching heavy vessels. The last launch of moment at this yard was that of the original Maine, in 1890, and this was hardly comparable, owing to the difference in size and weight, and also to the fact that the Maine was put overboard from a granite slip which had been tried many times, while the "Connecticut" was launched from a new and untried slip on ground none too certain.

A careful investigation of the best practice in battleship launching in this country and elsewhere was made with particular reference to the means employed in releasing the ship, in fastening the shoe pieces, greasing, stopping, and the methods of preventing accident in case of undue creeping after the shores and blocks had been removed, and before the time of sawing off or releasing, and of the allowable pressure on ground of various characteristics.

It was decided to adopt in general the standard Atlantic Coast practice, while recognizing the merit attached to that so successfully used at the Union Iron Works (see the paper abstracted in the second half of this article.—Ed.).

Without attempting to describe in detail the methods used on the "Connecticut," which do not vary essentially from the established Atlantic coast practice, it seems but right to call attention to the special features—first, the mask for retarding the speed after afloat; second, the dog shores, used as a preventer in case of creeping after removal of the shores and blocks; and third, the sand-bags used to replace certain keel blocks. All of the e were so successful that the writer would use them again under anything like similar circumstances, and would be inclined to extend the use of the third, a French idea originally, but adapted to this case by Naval Constructor W. J. Baxter, head of the Department of Construction and Repair, Navy Yard, New York.

A brief description of these items is as follows:

DOG SHORES.—Four dog shores, two on each side, located at frames Nos. 53 and 70, were provided to hold the ship should any weakness develop in the sole pieces. These shores were of 9 × 9-in. live oak, 7 ft. 6 ins. long, shod with 20-lb. steel plates on each end, the ends shaped so as to free themselves when a toggle underneath was removed and pressure applied from above. The lower or after end was rounded and rested against a 12 × 12-in. iron-shod live oak timber, set 3 ins. into the groundways and secured by 1-in. lag screws and blocked to ground for additional support. The forward or upper end rested against an iron-shod live oak timber 9 × 12 ins., set 3 ins. into the sliding ways, and secured by 1-in. lag screws, and blocked to the ship to prevent rising under pressure.

Each shore was held in position up to time of knocking out by a 6 × 6-in. shore at forward end, and a screw jack was fitted above to give pressure for removing if jammed, and a heavy sledge provided for emergencies.

MASK FOR CHECKING SHIP.—Owing to the crowded condition of the East River, it was desirable to prevent the "Connecticut" traveling too far or too rapidly, and, therefore, in order to check the speed of the ship, a mask constructed of 4½-in. yellow pine planks and backed by 12 × 12-in. yellow pine timbers was secured to the lugs on the stern post and to flanges on the ends of propeller shafts. It had an area of 155 sq. ft., the width tapering from 27 ft. at the bottom to 10 ft. at the top. The effect of the mask can be readily seen by an examination of the

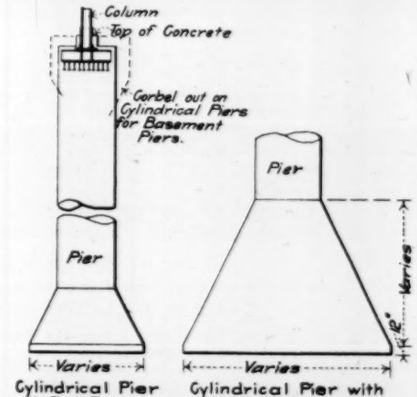


Fig. 2. Footings and Capping of Concrete Piers; First National Bank Building, Chicago.

velocity curve—the retarding effect up to the point of pivoting equalling closely the acceleration of gravity.

SAND-BAGS.—A few days before the launch the forward keel blocks from No. 5 to No. 16 inclusive were removed, and heavy canvas bags 6 × 12 × 24 ins., filled with dry sand, were substituted and wedged into place. These bags were double sewed at the seams, the seams being lapped in to give double canvas at the sewing. The bags were tested under a hydraulic press to a test pressure of 50 tons to insure strength, prior to installation. Just before starting to saw the sole pieces, these were easily

*Abstracts of two papers read before the Society of Naval Architects and Marine Engineers at New York, N. Y., November, 1904.

†Assistant Naval Constructor, U. S. Navy, New York Navy Yard, New York City.

and quickly removed by slitting open the ends with an axe, allowing the sand to run out.

The investigation mentioned above took the form, so far as possible, of plotting on the same scale the curves of actual results of similar launches; these curves, with tabular comparisons of such data as could not be plotted in the form of curves, put the data in such shape as to present immediately to the eye the points of similarity and the variations between ships, and to bring out the results which might reasonably be expected from a set of conditions approximating to those on which the curves and tables are based. This permitted a number of points to be decided with comparative ease which otherwise

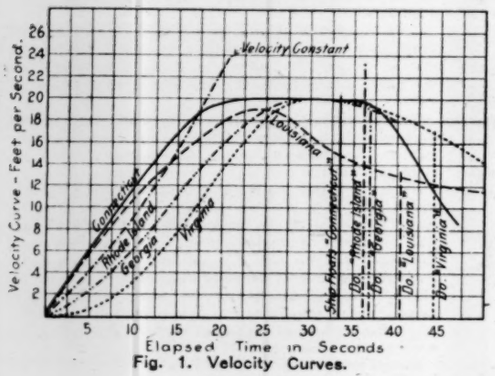


Fig. 1. Velocity Curves.

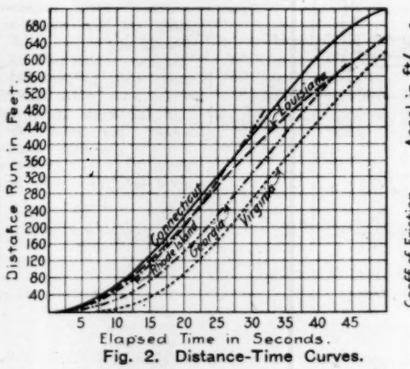


Fig. 2. Distance-Time Curves.

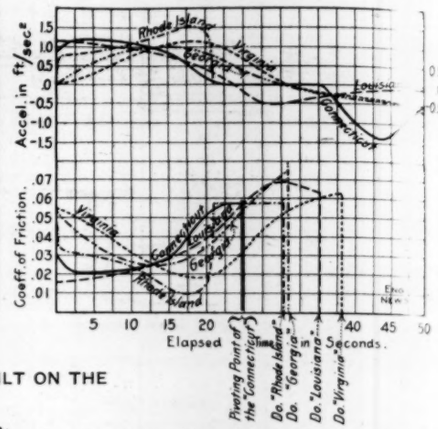


Fig. 3. Curves of Acceleration and Coefficient of Friction.

FIGS. 1 TO 3. LAUNCHING CURVES FOR SIX RECENT WAR-VESSELS BUILT ON THE ATLANTIC COAST.

would have been the subject of considerable laborious calculation.

Table I. herewith gives data which will assist in the determination of such points as the length, width and spread of ways, depth of water over the end, allowable pressure per square foot, number and size of wedges, number of rams, and the character of greases, and other general points which, taken in connection with the special conditions for a particular launching, may be of considerable assistance.

Fig. 1 shows the curves of velocity plotted to the same scale. It will be seen that the maximum velocity for the cases taken ranges from about 19 to 24 ft. per sec. The points on the curves where the ship floats are indicated, and the data thus obtainable in connection with the curves of distance, as shown in Fig. 2, would enable some determination to be made of the means necessary for limiting the travel after afloat, should the local conditions demand that such travel be limited, as is often the case. Table I. states the means actually adopted at the several yards for thus limiting the travel.

On Fig. 3 are shown the actual curves of coefficient of friction and acceleration of the cases taken, the coefficient of friction curves being plotted from the actual observed results, and from the formula:

$$C_f = \tan a - \frac{A}{g},$$

where

C_f = coefficient friction.
a = angle of ways with horizontal.
A = acceleration.
g = value of acceleration of gravity.

It will be seen that the gradient for launches of this character is about 21-32 to 11-16-in. to the foot, the launching weights varying from 5,500 to 7,000 tons, and the initial coefficients of friction run from about .03 to .05. These data with the launching weight to be expected for any particular case will obviously assist in the determination of the necessary fastening in the shoe piece if adopted, or the general character of any other type of releasing apparatus.

The table also gives the practice as to greases at the to be expected, and as far as practicable at several temperatures which might reasonably be expected at the date chosen for the launch. The greases adopted gave eminently satisfactory results, in that the launch was without hitch, the velocity good, and the grease on the groundways absolutely intact afterwards, no trouble whatever being experienced from undue heating.

The information from which the appended tables and curves were prepared was obtained from the Superintending Constructors at the yards at which the launches occurred, and the curves are all based on actual observations at the time of launching. I trust the information contained in them may be of interest and possibly of value in assisting the determination of certain points which must always trouble a yard man in preparing for the launch of a short, heavy ship of the battleship type.

RECENT LAUNCHING PRACTICE OF THE PACIFIC COAST.—By E. P. Lesley, Jun. Soc. N. A. & M. E.

The recent launches of the U. S. S. "California," "South Dakota" and "Milwaukee" from the shipyard of the Union Iron Works at San Francisco present some features that may be of interest. These three ships were built on piled slips with overhead traveling cranes similar to that described by Mr. James Dickie in a paper published in the Transactions of the Society of Naval Architects and Marine Engineers for 1900.

The arrangement of the launching cradle for the "California" is shown in the accompanying Fig. 4. This drawing explains itself and no description is needed. Particular attention, however, is invited to three features of this work: First, the character of the support for the groundways; second, the arrangement of crushing timbers under the fore poppets for distributing the pressure there when the stern lifts; and, third, the releasing device.

The groundways are supported above the floor of the building slips on blocks where the height is less than 6 ft., and beyond this point upon built-up A-frames. No blocks are fitted, the ways being made up to the proper height and crown by means of soft wood and shingles. No side shores are used except those necessary to correct irregularities in shape. The use of the soft wood and shingles insures a firm but elastic foundation since ways in the form of a perfectly true plane cannot be secured upon a rigid foundation without expense and much labor, and since all errors in the position of the ways are doubled as soon as the ship rises from the position of ramming up, it seems that a foundation strong enough to withstand the pressure that is calculated to have put upon it, and yet one that yields sufficiently so that at points where there is a tendency due to slight errors in shape, to exceed the calculated pressure, the lubricant will not be squeezed out, is greatly to be desired. That the kind of foundation here shown is safe is demonstrated by the fact that upon it

Mr. Dickie has launched a large number of ships and has had mishaps to none because of it. That it is easily and cheaply constructed is self evident, and that a practically uniform pressure on the ways is secured was clearly shown in the three launches under consideration by the even distribution of the grease upon the groundways after the ship had left them. There were no spots where they were rubbed dry and adjacent to them others where the grease remained thick.

The crushing timber method of distributing the pressure at the fore poppets was, the writer believes, originated by and is used only at the Union Iron Works. Experiments were made to determine the transverse crushing strength of Oregon pine,* with the following results: With grain of the wood at an angle of about 45°

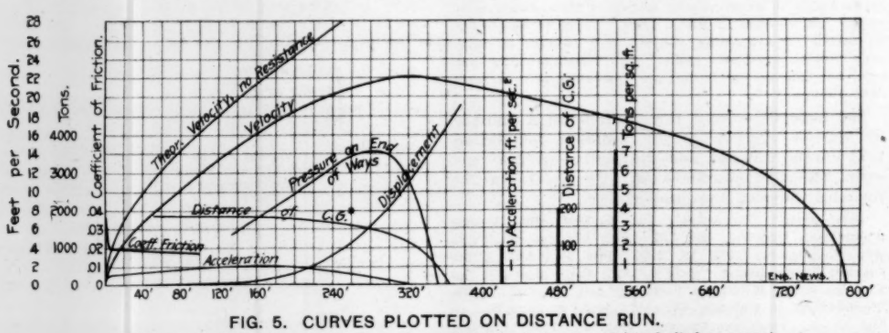


FIG. 5. CURVES PLOTTED ON DISTANCE RUN.

*This curve gives distance of center of gravity of land- borne weight abaft fore-poppet.

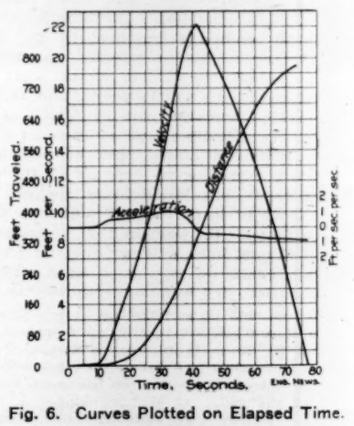


Fig. 6. Curves Plotted on Elapsed Time.

FIGS. 5 AND 6. LAUNCHING CURVES FOR U. S. S. "CALIFORNIA;" UNION IRON WORKS, SAN FRANCISCO, CAL.

*See table of properties of this wood in report of Annual Meeting of the Society of Naval Architects and Marine Engineers, Eng. News, Nov. 24, 1904, p. 483.

TABLE I.—LAUNCHING DATA FOR SIX RECENT WARSHIPS.

	CONNECTICUT. Battleship. Navy Yard, New York.	LOUISIANA. Battleship. Newport News, Va.	RHODE ISLAND. Battleship. Fore River, Quincy, Mass.	VIRGINIA. Battleship. Newport News, Va.	GEORGIA. Battleship. Bath, Me.	PENNSYLVANIA. Armored Cruiser. Cramp's, Philadelphia, Pa.
...e of warship						
... from which launched						
... of launch	Sept. 29, 1904.	Aug. 27, 1904.	May 17, 1904.	April 5, 1904.	Oct. 11, 1904.	Aug. 22, 1903.
...gth over all	456 ft. 4 ins.	456 ft. 4 ins.	441 ft. 6¾ ins.	441 ft. 3 ins.	441 ft. 1⅛ ins.	504 ft. 0⅜ in.
...gth between perpendiculars	450 ft.	450 ft.	435 ft.	435 ft.	435 ft.	502 ft.
...adth (extreme)	76 ft. 10 ins.	76 ft. 10 ins.	76 ft. 2 ins.	76 ft. 2⅛ ins.	76 ft. 1⅛ ins.	68 ft. 6¾ ins.
...el draft designed	24 ft. 6 ins.	24 ft. 6 ins.	23 ft. 9 ins.	23 ft. 9 ins.	23 ft. 9 ins.	24 ft. 1 in.
...placement at designed load draft	16,000 tons.	16,000 tons	14,948 tons	14,948 tons	14,963 tons	13,680 tons
...th (keel to main deck at side)	43 ft. 3 ins.	43 ft. 1⅝ ins.	42 ft. 4 ins.	41 ft. 3⅛ ins.	41 ft. 3⅛ ins.	41 ft.
... " upper "	50 ft. 9 ins.	50 ft. 7⅛ ins.	49 ft. 4½ ins.	.....	48 ft. 7⅛ ins.	48 ft. 6 ins.
...gth of ground ways (as erected)	498 ft.	.....	618 ft. 9 ins.	.....	510 ft. 6 ins.	490 ft.
" " (from sawing off to end)	486 ft. 6 ins.	529.72 ft.	582 ft. 3 ins.	528.25 ft.	488 ft. 6 ins.	488 ft. 4 ins.
" " (below M. H. W.)	178 ft. 6 ins.	155.83 ft.	138 ft. 2 ins.	154.68 ft.	163 ft.	78 ft.
" " (" M. L. W.)	108 ft.	105.75 ft.	0	99.11 ft.	26 ft.	0
...dth of ground ways	4 ft. 8 ins.	4 ft. 1 in.	3 ft. 11 ins	4 ft.	4 ft. 0½ in.	4 ft.
...ckness of ground ways	12 ins.	14½ ins.	11½ ins.	14½ ins.	12 ins.	12 ins.
...read " (center to center)	24 ft.	23 ft. 4 in. & 24 ft. 5½ in.	24 ft. 8 ins.	23 ft. 3½ in. & 24 ft. ¼ in.	24 ft. 2½ in. & 24 ft. 5½ in.	19 ft. 8 ins. and 20 ft.
...lination " per foot	¼ in.	⅝ in.	¼ in.	¾ in.	⅝ in.	¾ in.
" " transversely per foot	⅛ in.	¾ in.	⅛ in.	⅛ in.	0	⅛ in.
...mber " (in total length)	1 in.	0	0	0	19.2 ins.	⅛ in.
...gth of sliding ways (as erected)	380 ft.	374 ft. 6 ins.	364 ft.	.....	376 ft. 10 ins.	402 ft.
" (for pressure area)	360 ft.	371 ft.	342 ft.	351 ft.	360 ft.	365.1 ft.
...dth "	4 ft. 2 ins.	3 ft. 6 ins.	3 ft. 8 ins.	3 ft. 7 ins.	3 ft. 7 ins.	3 ft. 6 ins.
...read " (center to center)	24 ft.	23 ft. 9 in. & 24 ft. 10⅞ in.	24 ft. 11 ins.	23 ft. 4 in. & 24 ft. 2¼ in.	24 ft. 5 ins.	19 ft. 10 ins. and 20 ft.
...ickness of "	12 ins.	15⅛ ins.	11⅛ ins.	16 ins.	12 ins.	12 ins., 14 ins., 16 ins.
...a of sliding ways (effective)	3,000 sq. ft.	2,590 sq. ft.	2,701 sq. ft.	2,516 sq. ft.	2,586 sq. ft.	2,556 sq. ft.
...pth of water at end of ways (M. H. W.)	9 ft. 10 ins.	8.11 ft.	7.91 ft.	8.02 ft.	10 ft. 9 ins.	6 ft. 6 ins.
" " (M. L. W.)	4 ft. 11 ins.	5.51 ft.	0	5.42 ft.	21 ins.	14 ins.
" beyond end of ways (M.H.W.)	34 ft.	14.6 ft.	13 ft.	20.3 ft. av.	.....	30 ft.
" "	29 ft. 9 ins.	12.0 ft.	.....	17.8 ft. av.	.....	24 ft. 6 ins.
Elevation of after knuckle above (M. H. W.)	1 ft. 4 ins.	5.42 ft.	7.7 ft.	6.28 ft.	12 ins.	8 ins.
Inclination of keel blocks, per foot	⅜ in.	½ in.	½ in.	¼ in.	⅛ in.	⅛ ins.
Launching weight of hull and fittings	5,534.00 tons	6,114 tons	5,056 tons	5,447 tons	5,755 tons	5,300 tons
Steam engineering weights	601.00 "	578 "	644 "	219 "	1,015 "	127 "
Interior shoring, tools, dunnage, etc	91.00 "	83 "	110 "	70 "	213 "	253 "
Total launching weight of ship	6,226.00 "	6,774 "	5,810 "	5,737 "	6,988 "	5,680 "
Weight of cradle	315.00 "	266 "	184 "	255 "	250 "	200 "
Total weight of ship and cradle	6,541.00 "	7,040 "	5,984 "	5,992 "	7,233 "	5,890 "
Pressure on ways, tons per sq. ft.	2.18 "	2.72 "	2.22 "	2.38 "	2.80 "	2.30 "
Launching draft for'd	10 ft. 2½ ins.	11 ft. 1¼ ins.	11 ft.	10 ft. 0¼ in.	12 ft. 4 ins.	8 ft. 6 ins.
" aft	12 ft. 0½ ins.	13 ft. 2⅞ ins.	10 ft. 9 ins.	11 ft. 6⅛ ins.	13 ft.	13 ft. 5 ins.
" mean	11 ft. 1¼ ins.	12 ft. 1⅔ ins.	10 ft. 10⅜ ins.	10 ft. 9⅛ ins.	12 ft. 8 ins.	10 ft. 11⅛ ins.
Trim by stern	22 ins.	25 ins.	−3 ins.	1 ft. 6¼ ins.	8 ins.	4 ft. 11 ins.
Center of gravity of hull abaft amidships	4.35 ft.	3 ft. 11 ins.	−1.85 ft.	3.21 ft.	2.32 ft.	10 ft.
" above bottom of keel	21.27 ft.	.....	22 ft.	23.13 ft.	.....	20 ft.
" buoyancy of hull abaft amidships	.65 ft.	−1.15 ft.	−1.24 ft.	−1.33 ft.	−0.03 ft.	.....
" " above bottom of keel	6.2 ft.	6.58 ft.	5.89 ft.	6.16 ft.	7 ft. 2½ ins.	.....
Metacentric height	23.47 ft.	39.87 ft. (Rad.)	.....	25.62 ft.	35 ft. 5 ins. (Rad.)	15 ft. 7 ins.
Minimum moment against tipping, ft.-tons	270,000	51,000	.....	81,000	220,000	123,750
Distance traveled to pivoting point	326.5 ft.	437 ft.	435 ft.	420.3 ft.	338 ft.	345 ft. 4 ins.
Draft at A. P. when stern begins to lift	24 ft. 1 in.	18 ft. 11⅓ ins.	.....	18 ft. 4 ins.	26 ft. 9 ins.	22 ft.
Draft aft under cradle when lifting begins	.....	22 ft. 8⅓ ins.	.....	21 ft. 11 ins.	27 ft. 6 ins.	18 ft.
Pivoting pressure total	1,660 tons	1,296 tons	1,400 tons	1,258 tons	1,854 tons	1,180 tons
Length of fore poppets	7 ft. 3 ins.	.....	.....	.....	21 ft.	.....
Width "	4 ft. 2 ins.	.....	37 ins.	.....	12 ft. 8 ins.	.....
Area "	61.11 sq. ft.	.....	.....	.....	123 sq. ft.	.....
Maximum pressure on fore poppets per sq. ft.	27.2 tons	.....	23.2 tons	.....	15.05 tons	.....
Buoyancy when stern lifts, tons	4,881	5,400	4,410	4,625	5,180	4,378
Moment of weight about fore poppets, ft.-tons	1,135,622	1,175,000	972,000	984,000	257,000	1,095,000
Distance traveled when C. G. is at end of ways	288.15 ft.	340 ft.	417 ft.	342 ft.	304.2 ft.	260 ft.
" after knuckle enters water	20 ft. 6 ins.	103.95 ft.	134 ft. 3 ins.	114.98 ft.	.....	18 ft.
" when cradle enters water	0	99.45 ft.	153 ft. 6 ins.	18 ft. 6 ins.	.....	13 ft. 6 ins.
Distance from A. P. to end of ways	77 ft. 6 ins.	118 ft. 5 ins.	204 ft.	130 ft.	89 ft.	19 ft. 4 ins.
Ground ways at A. P. below M. H. W	.....	1.94 ft.	3 ft. 6 ins.	9.84 ft.	4 ft. 8 ins.	5 ft. 1 in.
Length of cradle remaining on ways when lifting begins	159 ft.	93 ft. 5 ins.	153 ft	101 ft. 10 ins.	144 ft.	110 ft.
Center of gravity beyond end of ways when lifting begins	28.35 ft.	97 ft. 1 in.	9 ins.	76.01 ft.	33.8 ft.	85 ft.
Height of keel at stern post above ground ways	.....	5.22 ft.	8 ft.	5.09 ft.	6 ft. 6 ins.	10 ft. 2 ins.
" fore foot "	7⅛ ins.	1.35 ft.	2 ft. 3 ins.	1.187 ft.	12 ft. 6 ins.	13 ins.
Calculated bodily drop of ship	12 ins.	.....	3 ft.	.....	4 ft.	4 ft. 8 ins.
Number of wedges on each side	226	270	379	257	248	402
Width of wedges	6 ins.	5 ins.	6 ins.	5 ins.	6 ins.	5 ins.
Pressure on each wedge, tons	14.41	13.04	7.84 tons	11.68	14.47	7.33
Size of saw log	6 in. x 50 in.	210 sq. ins.	210 sq. ins.	215 sq. ins.	370 sq. ins.	252 sq. ins.
Length of fastening sole piece to ground ways	10 ft.	.....	23 ft. 6 ins.	.....	16 ft. 10 ins.	12 ft.
" " " sliding "	10 ft.	.....	13 ft.	.....	.....	12 ft.
No. and size of bolts for fastening sole piece to ground ways, both sides	48, 1¼ ins.	45, 1¼ ins.	52, 1 in.	45, 1¼ ins.	54, 1 in.	54, 1 in.
No. and size of bolts for fastening sole piece to sliding ways, both sides	44, 1¼ ins.	58, 1⅛ ins.	58, 1 in.	58, 1¼ ins.	.....	54, 1 in.
Toggles forward on each side	1 double	2 single	4 single	4 single	0	4
" aft "	2 single	1 double	4 single	3 single	3	3
Lubricant on ways	3 single ⅛ in. stearine. ⅜ in. beef tallow, painted on hot. 1 coat graphite, sprinkled on.* ⅛ in. launching grease rubbed on.	4 single Ordinary tallow, first 150 ft. Hard tallow thence to pivoting point.† Extra hard tallow. Black oil.	5 single Tallow, ⅔. Lard oil, ⅓. Launching grease and black oil for 30 ft. from end of sliding ways.‡	12,224 lbs. tallow.† Black oil.	9,100 lbs. tallow. 126 " fish oil. 127 " soap.	5,841 lbs. tallow. No lard oil.
Nearest distance of ground ways to ship's bottom	23 ins.	30 ins.	.....	30 ins.	2 ft. 8 ins.	2 ft.
Number of dog shores on each side	2	0	0	0	.....	0
Size of dog shores	8 ins. x 8 ins. x 7 ft. 6 ins.	.....	.....	.....	.....	.....
Material of dog shores	Live oak	.....	.....	.....	Spruce	.....
Number of rams	62	72	106	70	35	.....
Area of shield	155 sq. ft.	0	0	0	0	0
Distance traveled when shield enters water	133.5 ft.	.....	.....	.....	.....	.....
No. and size of hydraulic jacks provided	2, 100-ton 2, 150-ton	4, 100-ton	4, 50-ton	4, 100-ton	2, 40-ton	.....

GENERAL LAUNCHING CONDITIONS.

Special conditions of ground, water and surroundings	Ground poor. Angle of building slip with shore line such as to give a good run up East River.	Character of ground solid. Building slip perpendicular to water front. Plenty of water room in front of slip.	Building slip perpendicular to shore line. Water room limited by bend in river.	Ground solid. Building slip perpendicular to shore line. Plenty of water room.	Building slip perpendicular to shore line. Sufficient water for a considerable run.	Ground good. Slip used before and known to be solid. Plenty of water at end of ways. Opposite slip a marshy island. Slip nearly perpendicular to shore line.
General features of ways	Ground ways rested upon a foundation of piles. Camber, 1 in. Sliding ways ordinary type. Cradle solid.	Ground ways supported in usual manner. No camber Sliding ways built in accordance with common practice. Cradle built solid.	Foundation for ground ways excellent, laid in cement. No camber. Sliding ways of ordinary type. Cradle built solid.	Ground ways supported in usual manner. No camber. Sliding ways of ordinary type. Cradle built solid.	Ground ways supported in usual manner. Camber of 19.2 ins. Sliding ways of ordinary type.	Ground ways supported in usual manner. Camber ⅛ in. Sliding ways ordinary type.
Precautions taken to prevent carrying away	Two sole-pieces of oak, sectional area each 300 sq. ins. Two dog shores on each side.	Two sole-pieces of oak, sectional area each 210 sq. ins. No dog shores.	Two sole-pieces of oak, sectional area each 210 sq. ins., with cross logs on under and upper sides. No dog shores.	Two sole-pieces of oak, sectional area each 215 sq. ins. No dog shores.	Two sole-pieces oak, sectional area each 370 sq. ins. No dog shores.	Two sole-pieces of oak, sectional area each 252 sq. ins. No dog shores.
Means adopted to check velocity when afloat	Shield, 155 sq. ft. area. It entered water after run of 133.5 ft.	None.	Two wedges drawn between heavy cross logs. Anchors were also used.	None.	Propellers blocked. Anchor dropped.	None.

* Applied between Sept. 8 and day of launch. † Applied six weeks prior to launch. ‡ Applied three weeks before launch and day before launch.

off flush with the top of the poppet board. Four were put in a straight athwartship line at the fore poppet (one over each way) and four were placed 24 ft. abaft this point. The distance they were pushed up through the poppet board showed the amount the timbers were crushed. On the "South Dakota" this was 7⅛ ins. at the fore poppet, tapering to nothing 28 ft. aft. Thus the pressure of 1,635 tons on the fore poppets when the stern lifted was distributed over a distance of 28 ft.

device was used it was most successful. The trip shore was knocked completely clear and the dog shore fell instantly. The arrangement of double ways makes the dog shore method of release most simple and easily applied.

Calculations were made, corrected for height of water, weight of ship and longitudinal position of the c. g. at the time of the launch for each ship. The curves drawn for the "California" are shown in Figs. 5 and 6. Particular attention is invited to the curve marked "pressure per sq.

nature of a series of springs so that the pre- exerted by them will be proportional to their def- tion. The first assumption, it is admitted, may not ar- ranted, but since some assumption must be m- this one is at least not the most unreasonable. Th- assumption is tenable, certainly, with ways sup- were those in the case of the launches under c- tion. Admitting these assumptions, the solutio- problem becomes simple.

Take, for example, the condition shown in th- Fig. 7. The ship is partially water borne and t- poppet has passed the end of the groundways. T- tion of the center of gravity of land-borne weig-

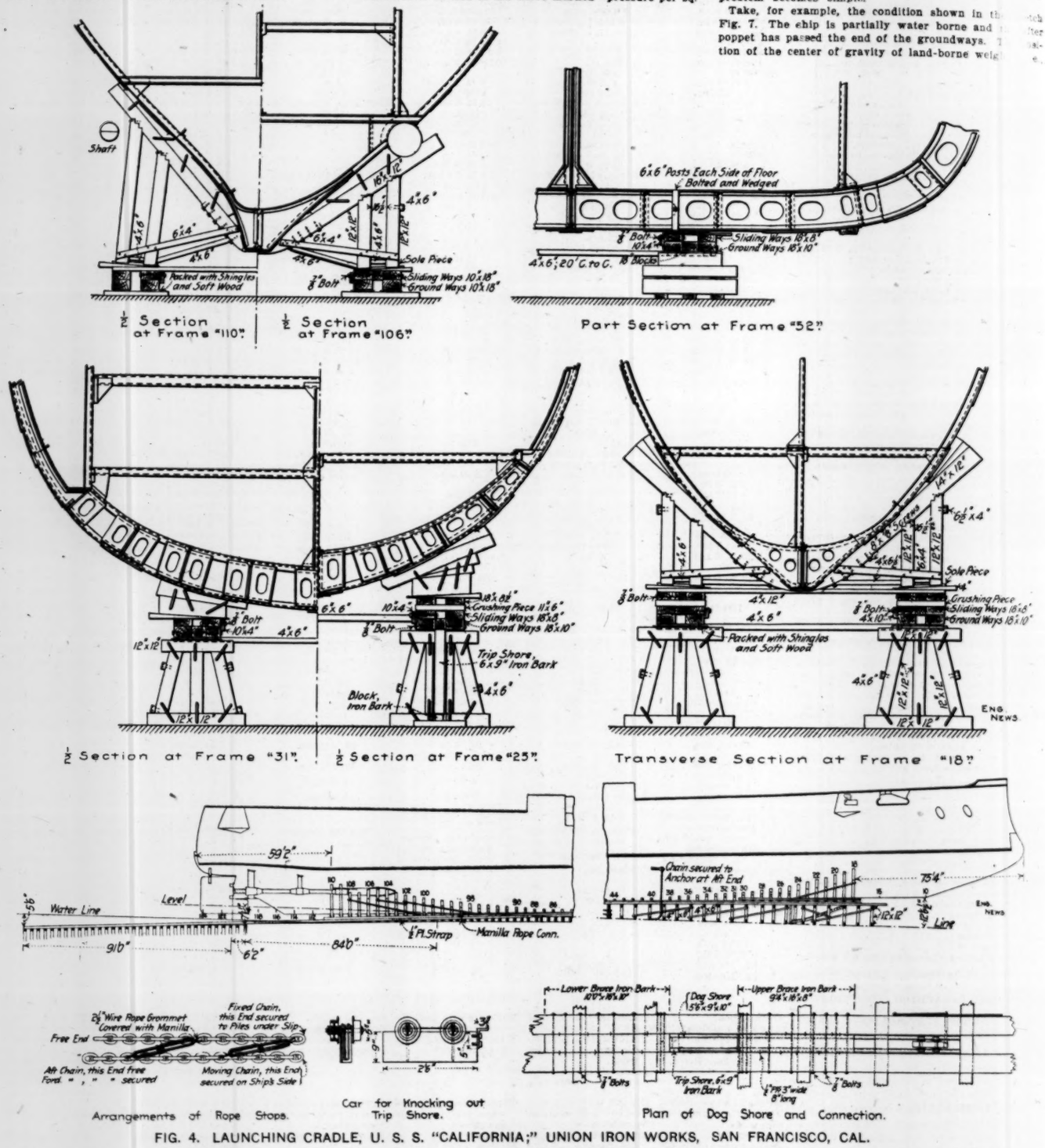

Shaft
Sole Piece
Packed with Shingles and Soft Wood
Sliding Ways 10"x18"
Ground Ways 10"x18"
½ Section at Frame "110"
½ Section at Frame "106"
6"x6" Posts Each Side of Floor Bolted and Wedged
Sliding Ways 18"x8"
Ground Ways 18"x10"
4"x6"; 20' C. to C. 18 Blocks
Part Section at Frame "52"
Crushing Piece 11"x6"
Trip Shore, 6"x9" Iron Bark
Block, Iron Bark
½ Section at Frame "31"
½ Section at Frame "25"
Transverse Section at Frame "18"
ENG. NEWS
59'2"
Water Line
Level
1" Pl. Strap
Manilla Rope Conn.
91'0"
62"
84'0"
Chain secured to Anchor at Aft End
75'4"
Lion
2½" Wire Rope Grommet Covered with Manilla
Free End
Fixed Chain, this End secured to Piles under Slip
Aft Chain, this End free
Moving Chain, this End secured on Ship's Side
Arrangements of Rope Stops.
Car for Knocking out Trip Shore.
Lower Brace Iron Bark 10"x18"x18'
Dog Shore 5"x8"x10"
Upper Brace Iron Bark 9"x18"x8'
Trip Shore, 6"x9" Iron Bark
Plan of Dog Shore and Connection.

FIG. 4. LAUNCHING CRADLE, U. S. S. "CALIFORNIA;" UNION IRON WORKS, SAN FRANCISCO, CAL.

Some trouble has been experienced with the ordinary arrangement of dog and trip shore released by a heavy pendulum, and the device shown in Fig. 4 was designed to obviate this. A car carrying a 300-lb. weight is held at the top of an inclined track (ending near Frame 25) by means of a trigger controlled from the launching platform. Being released, this car runs down the track and knocks the trip shore clear. The dog shore is iron shod and cut at an angle of 1 in 5 at the lower end. Immediately the trip shore is out the dog shore falls and the ship is free to move. In the three launches where this

ft. on end of ways." This curve is thought to be something new in launching calculations. It was pointed out to the writer by Mr. Dickie that the pressure per square foot upon the ship's bottom as it goes over the end of the ways, and not the moment to resist tipping should be the thing which determines the length of groundways; because some time before danger of tipping is reached this pressure may be sufficient to crush the bottom of the ship. In order to arrive at some determination of this pressure, two assumptions have been made; first, that the ship is rigid; and, second, that the groundways have the

the weight of ship less the displacement of the immerge- portion) may be computed. Let it be at G, a distan- B from the fore-poppet. The center of gravity of the re- action of the groundways must be in a vertical line wit- G. The length of the cradle remaining upon the wa- being L, the distribution of the reaction may be show- by a trapezoid whose length is L and whose center o- gravity lies at a distance B from the left-hand en-

The area of this trapezoid A is equal to the land-born- weight. The length of the ordinate Q at the right-han- end is the pressure per lineal foot at the end of the way-

...d of the ordinate P at the left-hand end the pres...re per lineal foot at the fore-poppet. When B is ...ater than $\frac{2L}{3}$ the figure becomes a triangle of ...ea A and base = 3 (L — B) and in this case the ...essure at the fore-poppet is zero. When B is less than $\frac{L}{3}$ the figure is again a triangle with area A and base ... This condition occurs just before the stern lifts and ...e pressure Q in this case is zero.

In order to avoid constructing the diagrams for different distances of travel, the writer has derived four formulas:

$$Q = \frac{2A}{L^2}(3B - L). \quad (1)$$

$$P = \frac{2A}{L^2}(2L - 3B). \quad (2)$$

$$Q = \frac{2A}{3(L - B)}. \quad (3)$$

$$P = \frac{2A}{3B}. \quad (4)$$

In which:
Q = pressure in tons per lineal foot at end of ways.
P = pressure in tons per lineal foot at fore-poppet.
A = land borne weight in tons.
L = length of cradle on the ways in feet.
B = distance of the center of gravity of land-borne weight from the fore-poppet in feet.

Formulas 1 and 2 are used when B is less than $\frac{2L}{3}$ but greater than $\frac{L}{3}$; formula 3 is used when B is greater than $\frac{2L}{3}$, and formula 4 is used when B is less than $\frac{L}{3}$. It may be noted in the case of formulas 1 and 2, that if B = $\frac{L}{2}$ then P = Q or the pressure upon the ways is uniformly distributed; but when B becomes greater than $\frac{L}{2}$, Q becomes greater than P. When B = L (formula 3) Q becomes infinite, this being the condition in which the c. g. of land-borne weight is at the end of the ways and the ship is balanced at that point.

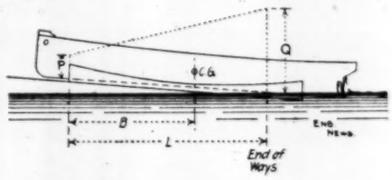


Fig. 7. Diagram of Position of Ship During Launch.

It is thought that although the deductions from these formulas may not be accurate, they are at least within reason and the results derived from their use will be of value for the purpose of making comparisons. According to good authority, in long ships of light construction, as are the "California," "South Dakota" and "Milwaukee," a pressure of from 7 to 9 tons per sq. ft. of sliding-ways is all that is safe on the bottom. For merchant ships with heavier plating and more closely-spaced frames probably from 15 to 20 tons may be reached with safety, but the limit of safety must needs be determined by working out the curves of pressure from varying actual conditions.

The velocity, acceleration and coefficient of friction curves (Figs. 5 and 6) were derived from a chronograph record of launching velocity.

The first of the ships launched, the "California," started very slowly, requiring 11 seconds to move the first foot. The "South Dakota" started much more quickly, requiring but 2¾ seconds to move the first foot. This was due probably to a less initial coefficient of friction because of a higher temperature and to a slightly greater inclination of the upper end of ways. The initial coefficient of friction for the "Milwaukee" was the smallest of all, and this ship started instantly. The very large quantity of soap used probably accounts for this. Owing to an accident to the chronograph no record of velocity for the "Milwaukee" was obtained after the ship had moved 13 ft., so that the velocity and friction data for this ship are somewhat meager. The writer has under construction a new chronograph, which is expected to be accident proof, and with which it is hoped to obtain data of launching velocity and friction in other yards under conditions different from those here presented.

A table of the launching data for these ships is here appended (Table II.). The writer desires here to thank Mr. James Dickie, the Superintendent of the Union Iron Works Shipyard, under whose supervision these vessels were launched; Mr. John G. Tawresey, the U. S. Naval Constructor, at whose direction the drawing of the launching cradle was made, and Mr. Leroy E. Coverly who made the calculations and drew the original curves and assisted in the deduction of the formulas, for their help in obtaining this data and their many courtesies during its accumulation.

TABLE II.—LAUNCHING DATA FOR THREE RECENT WAR-SHIPS.*

	California	S. Dakota	Milwaukee
Name of ship			
Where built	U. I. W.	U. I. W.	U. I. W.
Date of launching	April 28, 1904	July 21, 1904	Sept. 10, 1904
Length between perpendiculars	502 ft.	502 ft.	424 ft.
Length, normal L. W. L.	"	"	"
Beam, extreme	69 ft. 6½ ins.	69 ft. 6½ ins.	66 ft.
Displacement, normal, tons	13,700	13,700	9,700
Draught after launching, forward	10 ft. 7¾ ins.	11 ft. 1⅛ ins.	11 ft.
Draught after launching, aft	14 ft. 4¼ ins.	13 ft. 11⅛ ins.	14 ft. 4½ ins.
Draught aft when vessel lifts (pivots)	24 ft. 7 ins.	24 ft. 8 ins.	.....
Weight of ship, tons	5,980	6,108	4,650
Weight of cradle, etc., tons	82	82	76
Total moving weight, tons	6,062	6,190	4,726
Length of slidingways	369 ft.	369 ft.	324 ft.
Width of slidingways (effective)	34 ins.	34 ins.	34 ins.
Thickness of slidingways	Forward 8 ins. Aft 10 ins.	8 ins. 10 ins.	8 ins. 10 ins.
Area of slidingways, sq. ft.	2,091	2,091	1,828
Weight (ship and cradle) per sq. ft., tons	2.89	2.96	2.54
Slope of keel	1 in 27¾	1 in 27¾	1 in 26
Fore poppets from forward perpendicular	74 ft.	74 ft.	54 ft.
After poppets from after perpendicular	59 ft.	59 ft.	47 ft.
Length of groundways from fore poppets	503 ft.	505 ft.	409 ft.
Inclination, transverse	None	None	None
Slope, mean, for upper end	1 in 27.6	1 in 27	1 in 22
Slope, mean, for whole length	1 in 23.6	1 in 23.6	1 in 20.2
Camber	12 ins. in 300 ft.	11 ins. in 300 ft.	9 ins. in 264 ft.
Spread, center to center, upper end	20 ft.	20 ft.	25 ft.
Spread, center to center, lower end	"	"	"
Any additional out curve, extreme end	No	No	No
Thickness of groundways	10 ins.	10 ins.	10 ins.
How supported (piles, concrete, mud, blocks, cribs)	Piles	Piles	Piles
No. of piles (both sides)	672	672	644
Depth of water over end of ways	5 ft. 6 ins.	6 ft. 1 in.	7 ft. 3 ins.
Depth of water beyond end of ways	11 ft.	11 ft. 7 ins.	15 ft. 6 ins.
Calculated moment to prevent tipping	207,000 ft. tons	198,000 ft. tons.	.....
Calculated pressure on fore poppets when ship pivots	1,680 tons	1,635 tons	.....
Calculated pressure on fore poppets at end of ways	1,060 tons	1,033 tons	.....
Means for distributing pressure at fore poppets	Crushing pieces	Crushing pieces	Crushing pieces
Ship started instantly, promptly, slowly, very slowly	Very slowly	Promptly	Instantly
Time to move first foot (seconds)	11 sec.	2¾ sec.	1¾ sec.
Time to move ship's length	50 sec.	46.3	Not determined
Longitudinal position of center of gravity (ship) from fore poppets	185.3 ft.	183.6 ft.	161.6 ft.
Lubricant: Tallow	938 lbs.	1,150 lbs.	680 lbs.
Lubricant: Stearine	416 lbs.	420 lbs.	275 lbs.
Lubricant: Soft soap	929 lbs.	750 lbs.	790 lbs.
Temperature	53° F.	60° F.	68° F.
Coefficient friction, initial	.036	.0294	.0265
Coefficient friction, mean 50 ft. travel	.019	.0224	.0284
Velocity, maximum, ft. per sec.	22.1	19.23	Not determined
Travel before keel enters water	52 ft.	47 ft.	0 ft.
Travel to point of maximum velocity	320 ft.	310 ft.	Not determined
Total travel	786 ft.	783 ft.	622 ft.
Time to maximum velocity, seconds	41¼	35¼	Not determined
Total time, minutes	1 min. 16 sec.	1 min. 15 sec.	Not determined
Method of release	Dog shore	Dog shore	Dog shore
Estimated pressure on dog shore, total, neglecting friction	221 tons	224 tons	215 tons
Maximum estimated pressure per sq. ft. of slidingways at end of ways before stern lifts	7.2 tons	7.25 tons	.....
Means for checking:			
Rope stops broke	72	72	40
Total strength, each	30 tons	30 tons	30 tons
Anchors, number	2	2	2
Weight, each	16,500 lbs.	16,500 tons	16,500 lbs.
Anchors, distance dragged	35 ft.	34 ft.	47 ft.
Anchors dropped—buried	Buried	Buried	Buried
Other means for checking	Mud bank	Mud bank	Mud bank
Number of keel blocks	107	107	74
Number of other blocks and cribs	56	52	16
Number of shores	54	52	133
Number of packing wedges	186	186	72
Number of men employed on shore:			
Shipwrights	74	74	64
Riggers	10	9	8
Painters	17	12	10
Helpers, laborers, etc.	24	23	18
Time to ram up, minutes	30	25	21
Interval, minutes	15	20	17
Time to remove shores, minutes	14	11	21
Interval, minutes	21	24	12
Time to remove first half of blocks, minutes	29	29	18
Interval, minutes	12	12⅛	19
Time to remove second half of blocks, minutes	29	28⅜	21
Total time, including rests, from start to ram up until ship was released	2 hrs. 20 mins.	2 hrs. 20 mins.	2 hrs. 9 min.

* Corrected for actual conditions.

THE ENGINEER AND THE BUSINESS MAN.*

By J. Swinburne.†

In the Charter of the Institution of Civil Engineers the engineer is defined as "Directing the Great Sources of Power in Nature for the use and convenience of Man." With all respect to this august body, and their often-quoted definition, I would humbly suggest that it is inapt. It is really the definition of a scientific man. It is incomplete as applied to an engineer, because it does not take into account the sordid element of Price.

An American definition is much better: "An engineer is a man who can do for one dollar what any fool can do for two." This is not poetical, and is useless for oratorical purposes; but it is right. It is no use being able to design most complicated alternating-current machinery, or being able to explain it with the help of a wilderness of clock faces and several issues of the technical journals, unless the machinery, when made, is cheaper than its rivals.

Every design, every engineering manufacture, and every piece of engineering is only a question of price. It is unpleasant, perhaps, but it is a hard fact, and we have got to face it. If one of us does £150 worth of work a year, and earns £100, he is efficient; if he only does £90 worth, he is an unefficient machine, and will come to grief. He is like a 90KW. alternator which takes 100KW. to excite; though the analogy is not close. If he does £15,000 worth of work and gets £10,000, he is an efficient machine of much larger size, and his efficiency is much more satisfactory to himself.

I may mention, in passing, that an efficient man must do more work than he is paid for. This is not always realized. A man who only does what he is paid for would be of no use to the world at large. His efficiency is zero; his consumption being equal to his output. The man who does £15,000 worth of work and gets £10,000 consumes two-thirds of the work himself; so his efficiency is only 33%, which is very high, even for an engineer.

Science, for which no use has been found, or which is not applied, is called "Pure Science," whereas it is really the raw material, and should be called "Raw" or "crude science." There is an assumption of superiority in the term "Pure Science," and generally the term "Science" is appropriated by workers in raw science in much the same way as the term "working man" is appropriated to the exclusion of brain workers. There is supposed to be something noble and superior about "Raw Science," and its study is treated as the unselfish devotion to the interests of man, which is obviously entirely the wrong way round.

If you look round the electrical industry, or round the industries generally, who are at the top? Always the business men. The men at the head of large industries generally know very little science. A man may run a large electrical industry with the most vague ideas as to the true relation of the electrostatic and electromagnetic systems of units; in fact, he may think power, force and energy are very much the same kind of thing if looked

*Extract from an address before the students of the Institution of Electrical Engineers.

†Past-President, Institution of Electrical Engineers of Great Britain.

at in a broad common-sense way without any scientific prejudice.

If he wants good technologists he employs them. If he wants practical men who can take commercial responsibility he pays good salaries; if he wants men full of book knowledge he pays low salaries; but he does not generally want them. Raw scientists, to coin a horrible word for a most estimable class, tell him he knows nothing about science, and therefore does not know how to run a scientific business, and tell him to subscribe to universities (which are so inefficiently worked that they have to beg, like hospitals) and to employ young men from the technical colleges. He either takes no notice or he gets annoyed at their superior attitude, and discharges a couple of scientific men and puts their saved wages towards the salary of a new practical chauffeur, and enjoys himself.

Yet the science teacher looks down with contempt on the engineer as an ignorant, rule-of-thumb, inferior person, and the engineer in his turn looks down on the business man as a money-grubbing being with no brain and no lofty ideals. But this is all topsy-turvy. The business man is at the top, the practical engineer in the middle, and the unpractical engineer or the raw scientist at the bottom. The business man may have no knowledge of the ways of nature, but he has a knowledge of the ways of man, a knowledge which is infinitely more difficult to acquire and infinitely more difficult to employ well. His brain may be different from that of the scientific man; but there is no reason to suppose that it is less. Its convolutions may be different, but the probability is that they are even more complex than those of the scientific man.

A man's value to the world at large may generally be roughly estimated by the income he earns. Where position is earned at the same time, the money income is in proportion less for a given usefulness; but taking such disturbing elements into account, the rule is broadly true. The business man comes out far away above the engineer. He employs the engineer; the scientific man is his servant. Just as the raw scientist looks down on the engineer, and the engineer looks down on the business man, so the business man has a contempt for the engineer; and the engineer in his turn looks on the raw scientist as an unpractical crank.

So much is this the case that the business man will not trust the engineer more than he can help. He assumes that if you know anything about anything you cannot possibly be a business man. I remember a board of directors finding fault with a report of mine because I said that making a certain article would pay. They said such a statement was outside my province altogether, as I was a scientific man, and therefore could not possibly know whether a manufacture would pay, as that is a business question. As a matter of fact, I had at one time charge of a factory for making the article in question, but that did not matter; I was scientific, therefore it was not possible I could have any commercial sense. Now, how has such an idea come about? Is it not because scientific people profess such contempt for business that they do not trouble about it, and thus remain so useless that such ideas as those of my board are based on a foundation of truth.

If you examine the large industries you will, as I say, find the commercial or business man with little or no technical knowledge at the top of the tree. If you confine your attention to engineers, you will find the engineers who make the biggest incomes and occupy the most important and responsible positions are those who have most business or practical knowledge. Our leading consulting engineers do not spend a large portion of their lives plotting curves, counting electrons, or even making anything more than arithmetical calculations. They spend their time dealing with large questions on purely commercial lines; and as a rule the bigger the engineer the more he knows about practice and business, and the less he knows about text-book science. I do not for a moment mean to say that text-book science is not of priceless value; of course it is; and the more scientific knowledge you or I, or still more, the leading engineers have, the better; but most of us suffer from too little common sense in proportion to our scientific knowledge.

The engineers occupying smaller positions, assuming the same age in both cases, are not necessarily deficient in technical knowledge; but they are generally wanting in business attainment and less able to take responsible positions.

When I say that a man's earnings is a rough test of his value to the world, a great exception must be made in the case of genius. A genius does not work for a given employer; he works for the world at large, and the world at large does not pay him. It would be ludicrous nonsense to say that the value of Newton or Faraday could be reckoned in terms of their pecuniary earnings. They did grand work apparently because they were impelled to do it without any selfish motive. This is true of the great scientific men of to-day. We think now that we have no Newtons among us; the next generation will look back with envy to the present time as the day of great scientific men. The broad rule about measuring a man by his earnings breaks down, not only in the case of great geniuses, but to a proportionate extent in the case of many original workers in unapplied science.

We see, then, that the business man is the master; the engineer is his good slave; and the raw scientist is not good enough even to be the slave of the engineer; he has no market value at all, except as a teacher of more Raw Science. The raw scientist will remain at the bottom of the tree until he gets rid of the professional cant which pretends that raw science is pure, or nobler and superior to science as a whole; and the engineer will remain in the middle position as long as he takes the middle view and considers engineering as something superior to money considerations, and as long as he looks down on business and commercial methods.

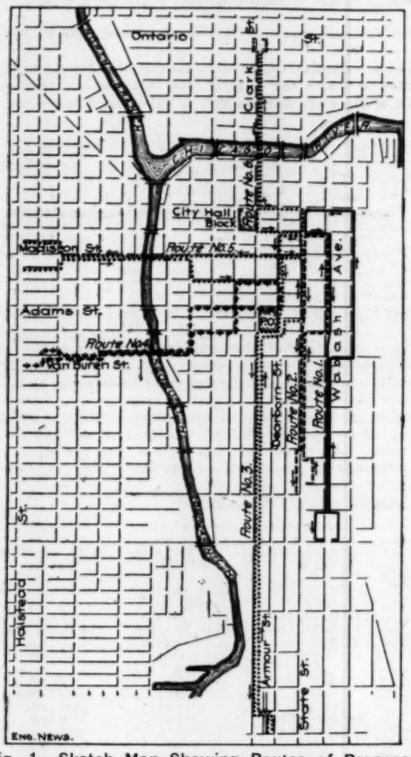


Fig. 1. Sketch Map Showing Routes of Proposed Underground Railway Lines in Chicago.

PROPOSED PASSENGER RAILWAY SUBWAYS CHICAGO, ILLINOIS.

A system of underground railways is pl[an]ned for the city of Chicago. In a recent report [the] Committee on Local Transportation of th[e] Chicago City Council, Mr. Geo. W. Jackson, [Con]sulting Engineer, recommends the abando[n]ment of the present river tunnels at Washington a[nd] La Salle Sts., the construction of double roa[d] bascule bridges at St. Clair, La Salle, Fra[nklin], Washington and Monroe Sts., and the prep[a]ration of plans for a comprehensive system of [ter]minal loops connecting the business section o[f the] city with its residence districts on the N[orth], South and West Sides. The removal of th[e] La Salle and Washington Sts. tunnels is estimate[d to] cost about $155,000, and the construction [of] the five new bridges is estimated to cost $2[,000,]000. The total cost of the proposed subways [will] be, it is thought, about $18,577,318. It w[ould] take according to the report about 30 mo[nths] to construct the subways.

The locations of the proposed subway rou[tes] are outlined on the accompanying map, Fig. 1. As will be seen, there are three lines extend[ing] from the business section of the city to the So[uth] side, two to the West side and one to the North side. Three of the six routes cross the river [re]quiring subaqueous tunneling. Roughly speaking, three forms of subway structure will be employed, a single track construction, a double track construction under streets, and a tunnel construction at the river crossings. Transverse sections showing these three types of structure are given in Figs. 2, 3 and 4. An important thing to be noticed is the provision made for pipe galleries in connection with the subway proper for passenger traffic. The further character of the work is best described in the words of Mr. Jackson's report, which is as follows:

In designing the several and different loops as shown on drawing Fig. 1, I have endeavored to accomplish the following practical purposes:

(1) An underground subway system making it practical and serviceable to put in operation the one-fare system without the issuing of transfers.

(2) An underground subway system without any so-called grade crossings at intersections.

(3) An underground subway system which becomes practical for cars from the different divisions of the city to be routed to whatever division of the city they may be directed to without grade crossings.

(4) An underground subway system which provides a 10 ft. walk its entire length, enabling passengers and pedestrians to walk underground to any station they might desire to reach without coming to the surface, as well as having a continuous landing for passengers to leave or gain access to cars and preventing congestion.

(5) An underground subway system which makes it practical to install and operate on large radius curves at intersections, and at the same time have the necessary room for the placing of the different corporation utilities which become a very important factor in designing a serviceable and practical subway for the accommodation of street cars and corporations' and city's utilities.

(6) An underground subway system which provides practical space for the placing of appliances of the different corporations operating under license from the City Council as well as space for new utilities which the city government may wish to install, and at the same time leave space enough for the future development of our city to an extent way beyond the most sanguine hope of the growth of the city.

(7) An underground subway system which would take

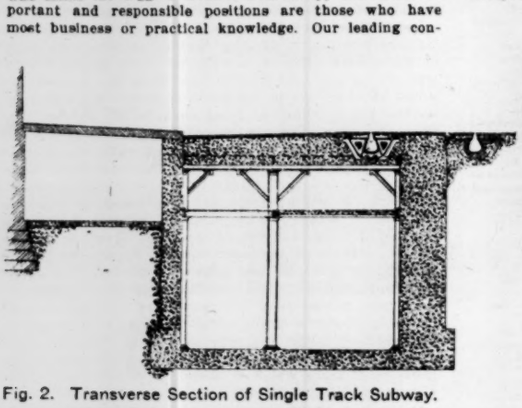


Fig. 2. Transverse Section of Single Track Subway.

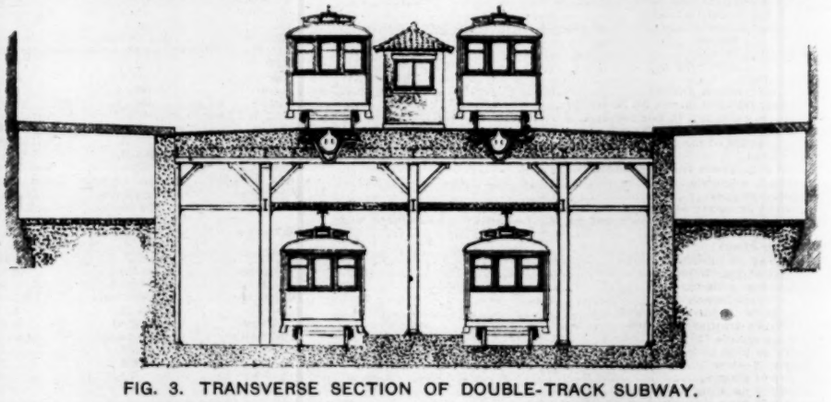


FIG. 3. TRANSVERSE SECTION OF DOUBLE-TRACK SUBWAY.

...s of our present unsanitary sewers as well as make ...vision for the taking care of new sewer systems in ...section with a high pressure water system which ...ld be built at once in connection with a new storm ...r sewer system, as shown on drawing, Fig. 3, and ... making provision for the economical and speedy way ...making connection from the different corporation sys... to buildings along the line of subways.

...) An underground subway system which will not re... the city or property owners going to the large ex... of removing the different appliances which have ...en installed under our sidewalks, as well as taking ...re of and making it practical and possible to construct ... subway without the city assuming a possible large ...bility caused by the settlement of buildings during the ...nstruction of subway.

Route No. 1, which is shown on drawing, Fig. 1, is de...gned with entrance beginning at 12th St. and Wabash ...ve., running east and west to the rear end of the prop...ty facing Wabash Ave., continuing north to Harmon ...ourt and east and west on Harmon Ct., to Wabash Ave. ...A cross-section of the subway as designed is shown on ...rawing, Fig. 4. The purpose of designing the entrance ...nd exit to the subway, as shown on drawing, Fig. 1, is ...o accomplish the purpose of not destroying the streets ...at the entrance of the subway. The property necessary ...to be acquired for the construction of the subway en-

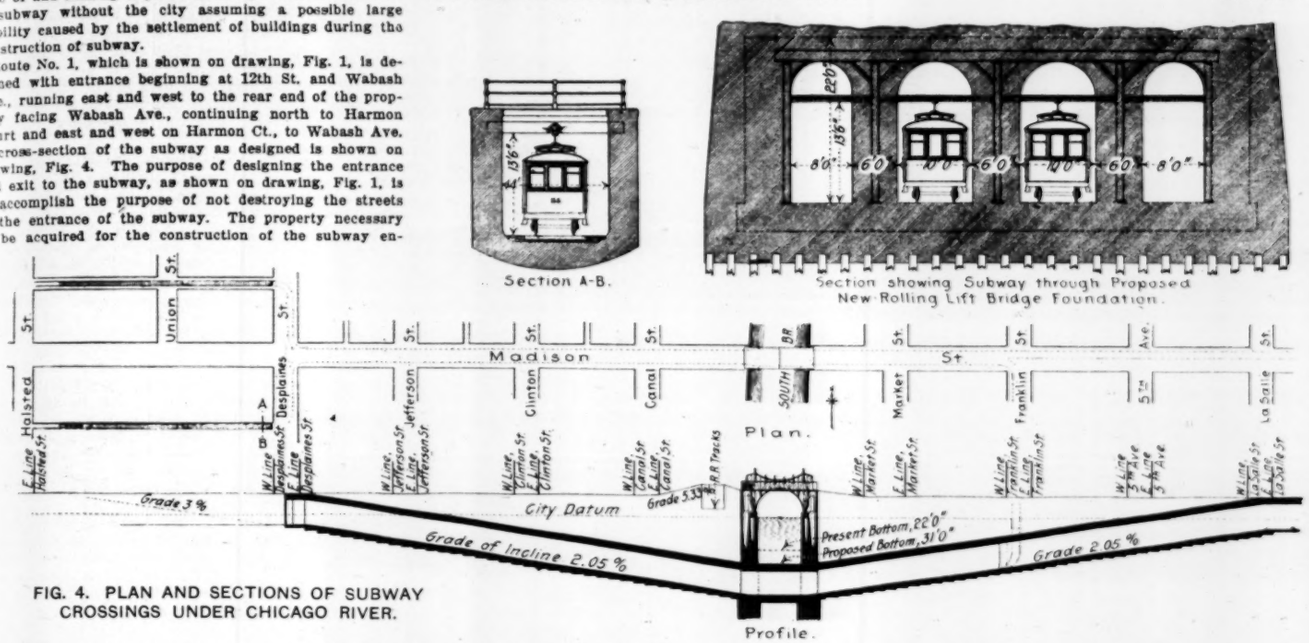


FIG. 4. PLAN AND SECTIONS OF SUBWAY CROSSINGS UNDER CHICAGO RIVER.

trances and exits would not necessarily be expensive, as the upper part of the property could be leased for any purpose desirable. From Harmon Ct. a double track subway north on Wabash Ave. to Van Buren St., as shown on drawing, Fig. 2; single loop subway east on Van Buren St. to Michigan Ave.; north on Michigan Ave. to Randolph St.; west on Randolph St. to State St., as shown on drawing, Fig. 3; south on State St. via double-track system to Washington St., east on Washington St. to Wabash Ave., south on Wabash Ave. to Jackson Boulevard, as shown on drawing, Fig. 3; south on Wabash Ave. from Jackson Boulevard to Van Buren St. via single-track subway, as shown on drawing, Fig. 2.

Route No. 2, which is shown in drawing, Fig. 1, entrance to be on State St. and Eldridge Ct., running east to the rear end of the property facing west of State St., thence north to Peck Ct., west on Peck Ct. to State St., thence north on State St. to Polk St. This part of the subway is designed as shown on drawing, Fig. 4. North on State St. from Polk via double-track subway to Jackson Boulevard, as shown on drawing, Fig. 3; east on Jackson Boulevard via single-track subway to Wabash Ave., as shown on drawing, Fig. 2; thence north on Wabash Ave. via double-track subway to Washington St., thence west on Washington St. to State St., thence south on State St. to Quincy St., as shown on drawing, Fig. 3; thence south on State St. to Jackson Boulevard via single track, as shown on drawing, Fig. 2. Exit to be from State St. west in Polk to rear end of property facing east on State St., thence south to Taylor St. via single track, as shown on drawing, Fig. 4.

Route No. 3, which is shown on drawing, Fig. 1, is designed with an entrance beginning at 18th St. on private property between Armour and Dearborn Sts. via double track north of 17th St. and thence west on 17th St to Clark st.; thence north on Clark St. via double track to Jackson Boulevard, east on Jackson Boulevard to Dearborn St., north on Dearborn St. to Quincy St., as shown on drawing, Fig. 3; east on Quincy St. via single track to State St., as shown on drawing, Fig. 2, north on State St. via double track to Madison St., west on Madison St. to Dearborn St., south on Dearborn St. to Adams St., as shown on drawing, Fig. 3, thence south on Dearborn St. via single track to Quincy St., as shown on drawing, Fig. 2.

Route No. 4, which is shown on drawing Fig. 1 and marked with green, is designed with entrance on Halsted St. in rear of property facing north on Van Buren via single track east to a point about 275 ft. west of Desplaines St., thence north to Van Buren St. (across section of the subway as designed is shown on drawing, Fig 4), thence east on Van Buren St. via single track to Desplaines St., as shown on drawing, Fig. 2, thence via double track to Clinton St., thence north on Clinton St. to private right of way, thence east to Franklin St., thence north to Jackson Boulevard, as shown on drawing, Fig. 3, thence east on Jackson Boulevard via single track to La Salle St., thence north on La Salle St. to Adams St., thence east on Adams St. to Dearborn St., as shown on drawing, Fig. 2, thence north on Dearborn St. via double track to Monroe St., thence west on Monroe St. to La Salle St., as shown on drawing, Fig. 3, thence south on La Salle St. via single track to Adams St., west on Adams St. to Franklin St., south on Franklin St. to Jackson Boulevard, as shown on drawing, Fig. 2, exit to be north on Desplaines St. from Van Buren via single track to a point 95 ft. north of the north line of Van Buren St., thence west to private property to a point 400 ft. west of the west line of Desplaines St., thence south to Van Buren St. A cross-section of exit is shown on drawing Fig. 4.

Route No. 5, which is shown on drawing, Fig. 1, is designed with entrance on Halsted St. north and south of Madison St., thence east to Desplaines St., thence north and south on Desplaines St. to Madison St. (cross-section of the subway as designed is shown on drawing, Fig. 4), thence east via double track on Madison St. to Franklin St., as shown on drawing, Fig. 4, thence south on Franklin St. via single track to Monroe St., thence east on Monroe St. to La Salle St., as shown on drawing, Fig. 2, thence east via double track to Dearborn St., thence north on Dearborn St. to Madison, thence east on Madison to State St., thence north on State St. to Washington St., thence west on Washington St. to Dearborn St., as shown on drawing, Fig. 3, thence south on Dearborn St. via single track to Madison St., thence west on Madison St. to Franklin St., as shown on drawing, Fig. 2.

Route No. 6, which is shown on drawing, Fig. 1, is designed with entrance beginning at Clark and Ontario Sts., running east and west to the rear end of property facing on Clark St., continuing south to Ohio St. and east and west on Ohio St. to Clark St. A cross-section of the subway as designed is shown on drawing, Fig. 4. From Ohio St. a double track running south on Clark St. to Randolph St, thence south on Clark St. via single track to Washington St., thence east via single track on Washington St. to Dearborn St., as shown on drawing, Fig. 2, thence east via double track on Washington St. to State St., thence north on State St. to Randolph St., as shown on drawing Fig. 3, thence west on Randolph St. via single track to Clark St., as shown on drawing, Fig. 2.

ESTIMATE COST OF CONSTRUCTION OF SUBWAY.

Route No. 1.

1,500 lin. ft. of single track approach at $62..	$ 93,000.00
4,989 lin. ft. of single track subway at $185..	922,965.00
4,132 lin. ft. of double track subway at $426..	1,760,232.00
	$2,776,197.00

Route No. 2.

1,278 lin. ft. of single track approach at $62..	$ 79,050.00
1,037 lin. ft. of single track subway at $185..	191,845.00
4,053 lin. ft. of double track subway at $426..	1,726,578.00
	$1,907,473.00

Route No 3.

400 lin. ft. of double track approach at $134.	$ 53,600.00
653 lin. ft. of single track subway at $185..	120,805.00
9,148 lin. ft. of double track subway at $426..	3,897,048.00
	$4,071,453.00

Route No 4.

1,120 lin. ft. of single track approach at $62..	$ 69,440.00
4,070 lin. ft. of single track subway at $185..	752,950.00
3,284 lin. ft. of double-track subway at $426..	1,398,984.00
172 lin. ft. of double track subway crossing river at $1,001	172,172.00
	$2,393,546.00

Route No. 5.

1,660 lin. ft. of single track approach at $62..	$ 102,920.00
3,764 lin. ft. of single track subway at $185..	696,340.00
3,475 lin. ft. of double track subway at $426..	1,480,350.00
197 lin. ft. of double track subway crossing river at $1,001	197,197.00
	$2,476,807.00

Route No. 6.

794 lin. ft. of single track approach at $62..	$ 49,228.00
1,682 lin. ft. of single track subway at $185..	311,170.00
3,073 lin. ft. of double track subway at $426..	1,309,098.00
225 lin. ft. of double track subway crossing river at $1,001	225,225.00
	$1,894,721.00

Total cost of 50,703 lin. ft. of subway including all routes with all the necessary appurtenances, as shown on drawing, Fig. 3	$15,610,197.00
Purchase of property	1,250,000.00
Engineering & Omnibus Expenses 7% of the total	1,092,713.79
Incidental expenses, labor, insurance, accidents and possible damages 4%	624,407.88
Grand Total	$18,577,318.67

In estimating the cost of the construction of the different cross-sections of subways I have taken into consideration the construction of stations for the entrance and exit of passengers, stations to be placed not to exceed 450 ft. apart, as well as the repaving of the streets and the construction of a practical underground conduit system for trolley purposes, as shown on drawing, Fig. 3, the placing of the different corporation pipes and cables which now occupy space in the streets in which subway would be constructed, as well as taking care of the city's present utilities and including the building of a new sub-soil and storm water system and high pressure water pipe systems, all of which is covered in the above estimates.

In conclusion I beg leave to say that in my judgment the construction of the 50,703 lin. ft. of subway and all of the necessary appurtenances, as shown on drawing, Fig. 3, can with the proper executive ability be completed in 30 months' time without discommoding the general public to any great extent.

THE ANCHORAGES FOR THE MANHATTAN BRIDGE AT NEW YORK CITY.

Work on the revised plans for the third suspension bridge across the East River at New York City has progressed to the point where bids are being asked for the construction of the anchorages. As many of our readers know, the original plans calling for a chain cable suspension bridge at this location were revised, and a wire cable bridge was substituted, the preliminary plans for which were published in Engineering News of July 7, 1904. The present drawings are the final development of these preliminary plans so far as the anchorages are concerned, and from

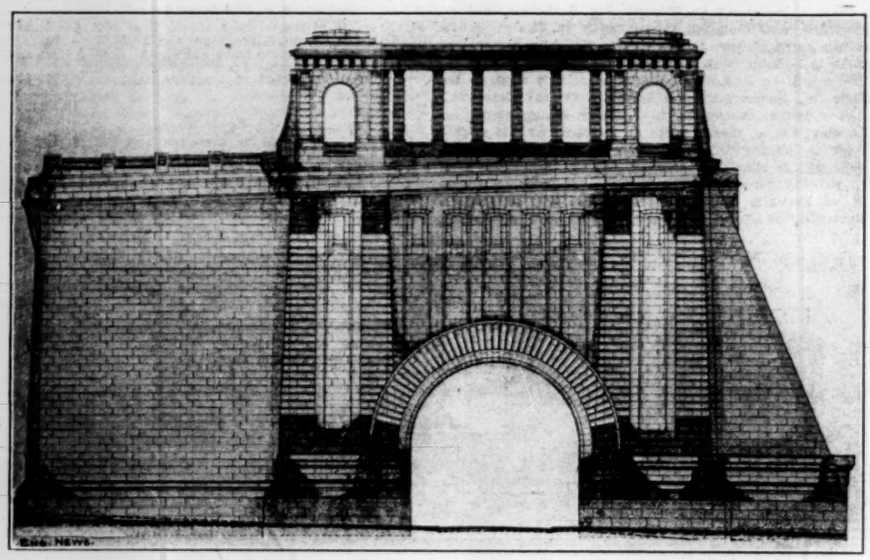


FIG. 1. SIDE ELEVATION OF ANCHORAGE FOR MANHATTAN SUSPENSION BRIDGE.

them we have chosen only those necessary to give a clear idea of the anchorage construction.

GENERAL DESCRIPTION.—The axis of the Manhattan Bridge makes an angle of about 22½° from the meridian and the anchorages are centered on this line at a distance apart between faces of about 2,890 ft. The two anchorages are practically identical in construction. Each consists of a rectangular mass of masonry pierced transversely by an archway for street passage and longitudinally by an arched footway at right angles to the main arch, and extending through the forward abutment. These arched passages are at street level and above their crowns the masonry is of cellular construction. The masonry is of concrete and rubble concrete with all exterior surfaces faced with cut stone. Its general appearance is shown by the side elevation, Fig. 1, and the half front and half rear elevation, Figs. 2 and 3.

The construction of the anchorage is best explained by the accompanying sectional drawings. Fig. 4 shows a longitudinal section on the line of the cables, and explains the construction of the anchors proper. The chains, reaction girders and other members composing the anchors are shown in detail by Fig. 5. Fig. 6 shows a longitudinal section on the center line of the anchorage, and Fig. 7 shows transverse sections through the front and rear abutments, respectively. Referring to Figs. 4 to 6, generally it will be noticed that the masonry is rubble concrete except where reinforcement is introduced and except, of course, for the granite facing. Other points to be noted are the cellular construction and the granite pedestals for the cables. An interesting feature of the work is furnished by the drawings of Fig. 6. These show the wind anchorages, the stiffening truss anchorages and the foot bridge anchorages.

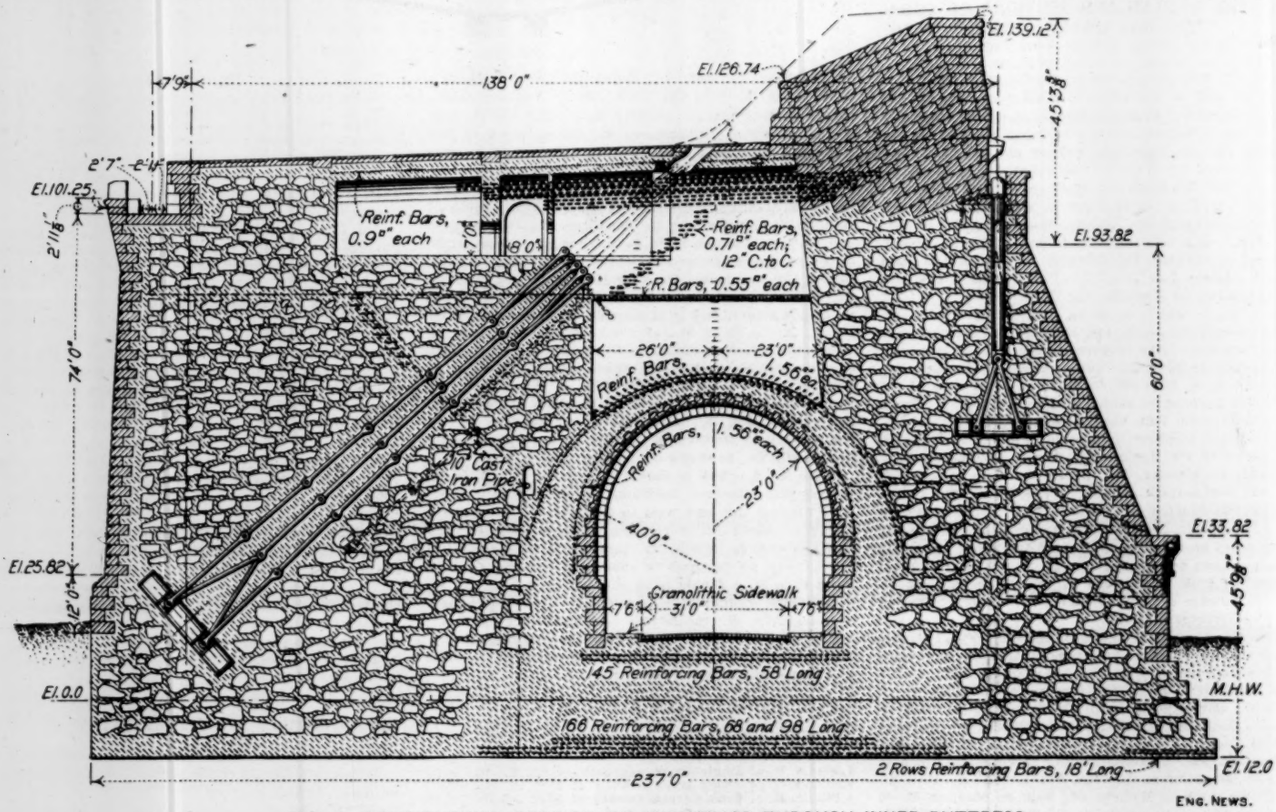


FIG. 4. LONGITUDINAL SECTION OF ANCHORAGE THROUGH INNER BUTTRESS.

The foot bridge anchorages are for tempor[illegible]se only, but the wind and truss anchorages a[illegible]er-manent. The wind anchorages are located [illegible]the center of the front face of the masonry a[illegible]or-age, and there is a truss anchorage under e[illegible]of the four stiffening strusses.

MATERIALS AND CONSTRUCTION.—The character of construction called for is be[illegible]x-plained by the specifications the principal [illegible]ts of which are given as follows:

EXCAVATION AND PILES.—It is expected th[illegible]he excavation for the foundation will be made ap[illegible]t-mately to the depth of 12 ft. below mean high wat[illegible]he lump sum of the contract being based on the quanti[illegible]of excavation and material required to go to this depth. [illegible]ut should conditions develop during the progress of th[illegible]x-cavation that render it necessary or desirable to go d[illegible]er or not so deep, the depths may be changed as requir[illegible]by the engineer.

The sides of the excavation for the anchorage sh[illegible]be so held by sheet piling and adequately braced that the clear space required inside for the foundation ca[illegible]be maintained, and the sides of the cut prevented from cav-ing. The sheet piling shall follow the outline of the foundation with the inner dimension of inclosed area to provide for the contingency of carrying the foundation to a maximum depth of 21 ft. below mean high water, and increasing the area of base of anchorage. The sheet piling shall consist of 12 × 12-in. yellow pine, dressed on all faces, grooved and splined together, or other sheet piling in the opinion of the engineer equally good. It shall be driven to at least 15 ft. below mean high water, with provision for going to a maximum depth of 24 ft., and shall remain permanently in place, with the top extending not to exceed 7 ft. above mean high water. The mean high water datum referred to in these specifications is that established by the United States Government at the Brooklyn Navy Yard.

When the excavation for the foundation has reached the depth at which the engineer deems it desirable to test the bearing value of the soil, the contractor shall drive test piles at such points as may be directed by the engineer. These test piles shall not be more than ten in number, and shall be at least 30 ft. long, with a diameter of at least 14 ins. at the head, and 9 ins. at the point. They shall be driven to such depth and in such manner as the engineer may direct, and the penetrations carefully noted. The weight of hammer used shall be 4,000 lbs. Simultaneously with the pile tests, the contractor shall at five points designated by the engineer set in the ground, at a depth to insure resting on undisturbed material, posts 2 ft. square, and load these with known weights until a distinct settlement is observed. From the results of the

ing tests it shall be decided whether piles be used, and if so, the number, spacing and length of piles required.

...s, if used, will be driven to a depth of about 25 ft. ... the bottom of the foundation, or to such depth as ... engineer may direct, and will project 2 ft. or 1 ft. ... the bottom, as shown ... ...ns. If required by the ...neer, the water jet shall ... used to force piles to ... required depth. All piles ... be banded or other... protected against split... or brooming, and where ...essary to facilitate driv... shall be shod. The parts ... the piles projecting above ... bottom of the foundation ... have the broomed tops ...ed off, and be trimmed of ... bark and loose wood be... ... concrete is laid about ... After the foundations ... carried to sufficient ...ight, the space about the ...horage shall be filled with ...lected material, thoroughly ...t and rammed.

All excavated material shall be removed from the anchorage site as it is taken out, except such as may be selected for back-filling or filling cellars within the area of bridge property. The excavation shall be kept sufficiently free from water to enable all operations to be properly conducted, and in order that all concrete shall be laid dry. After the excavation is completed and the piles driven, and before the laying of the concrete is begun, the bottom of the excavation throughout shall be so rammed as to thoroughly compact the material. Should it become necessary to carry different parts of the foundations to different depths, the bottom of the excavations must be made level, and where the change from one elevation to another occurs, the vertical face must be securely held by sheet piling and bracing. All piles must be driven in the lower levels, and these filled with concrete ... the elevation of the upper levels before the piles are driven in the upper levels.

PILES.—All piles used shall be spruce or long-leaf yellow pine, straight, sound, free from shakes or large knots, and not less than 14 ins. in diameter at the head and 9 ins. at the point.

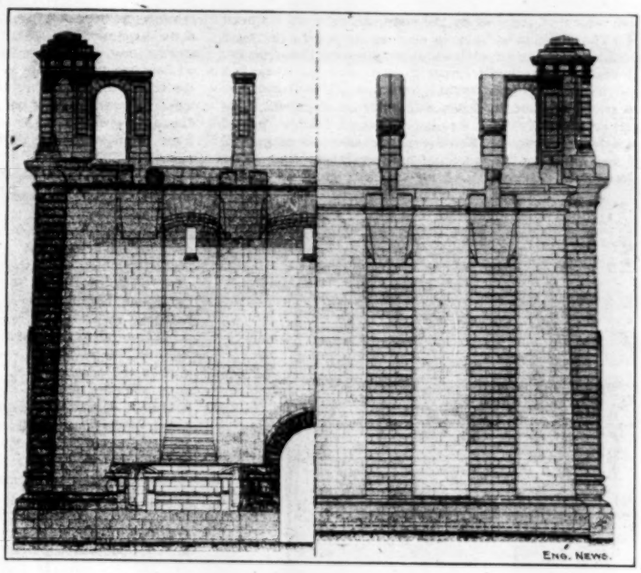


Fig. 2. Half-Front Elevation. Fig. 3. Half-Rear Elevation.

FIGS. 2 AND 3. FRONT AND REAR ELEVATIONS OF ANCHORAGE FOR MANHATTAN BRIDGE.

CEMENT.—All cement used in this work must be of the best quality of imported or American Portland and manufactured by works of established reputation for furnishing a cement of high grade and uniform quality, and which have been in successful operation for at least two years. The cement must have such chemical composition as will be satisfactory to the engineer, based on tests by chemists selected by him.

The cement shall be sufficiently fresh to have lost no strength from age, but must not be so fresh as to be "hot" and quick setting. It must be dry and free from lumps when used. The cement shall be delivered either in sacks or barrels, plainly marked with the manufacturer's brand. It shall be stored in a dry place, free from air currents or other source of injury, and so piled as to give convenient access to every carload lot, or to every 40-ton lot, when not delivered in cars. The contractor must provide adequate storage and enough cement ahead to enable seven-day tests to be made before cement shall be used, and not less than ten days (holidays and Sundays excluded) must elapse between the time the samples are drawn and the time the cement is used.

Samples of cement for testing shall be taken in such manner as the engineer may direct, and the tests shall be made under his immediate direction. The requirements hereinafter specified shall apply to each carload lot, or its equivalent of 40 tons, and failure to meet those requirements shall be cause for rejection of such lot. Each of the tests herein specified shall be applied to each carload of 40-ton lot of cement, and for the tension tests at least one sample will be taken from each 5 tons of cement; from each sample six briquettes will be made, three without sand and three containing one part of cement to three parts of standard crushed quartz, by weight. The tests will be made on the individual samples, without intermixing.

The cement shall be so finely ground that at least 90% will pass through a sieve having 10,000 openings per square inch, and at least 75% through a sieve having 40,000 meshes per square inch, the wires of the sieve being respectively 0.0045 and 0.0024-in. diameter. The specific gravity of the cement, determined from a sample which has been carefully dried, shall be between 3.10 and 3.25.

The cement shall not acquire its initial set in less than 30 minutes, and must have acquired its final set in 2½ hours. The pats made to test its soundness may be used in determining the time of setting. The cement is considered to have acquired its initial set when the pat will bear, without being appreciably indented, a wire 1-12-in. in diameter, loaded to weigh ¼ lb., and the final set when the pat will bear, without being appreciably indented, a wire 1-24-in. in diameter, loaded to weigh 1 lb.

For a test as to the soundness of the product at least two pats of neat cement, mixed for five minutes with 20% of water by weight, shall be made on glass, each pat about 3 ins. in diameter and ½-in. thick at the center, and tapered to a thin edge. The pats shall be kept under a wet cloth until thoroughly set, when one shall be placed in fresh water, maintained at a temperature of about 70° F. for 28 days, and the other on a rack in the upper part of a closed vessel partly filled with water, maintained at a temperature of about 212° F. After having been thus exposed for six hours, the cake shall be immersed in the water in the vessel for 18 hours, the same temperature being maintained. Neither of the cakes thus tested shall show cracks, blowing or warping.

The sand to be used with the cement for testing purposes shall be the standard crushed quartz, of such fineness that it will pass through a sieve having 400 openings per square inch, and be caught on a sieve with 900 openings per square inch, the diameter of the wires to be one-half the width of the openings. Mortar for tests shall be mixed by hand. The proper amounts of cement, sand and water shall be determined by weight. The temperature of

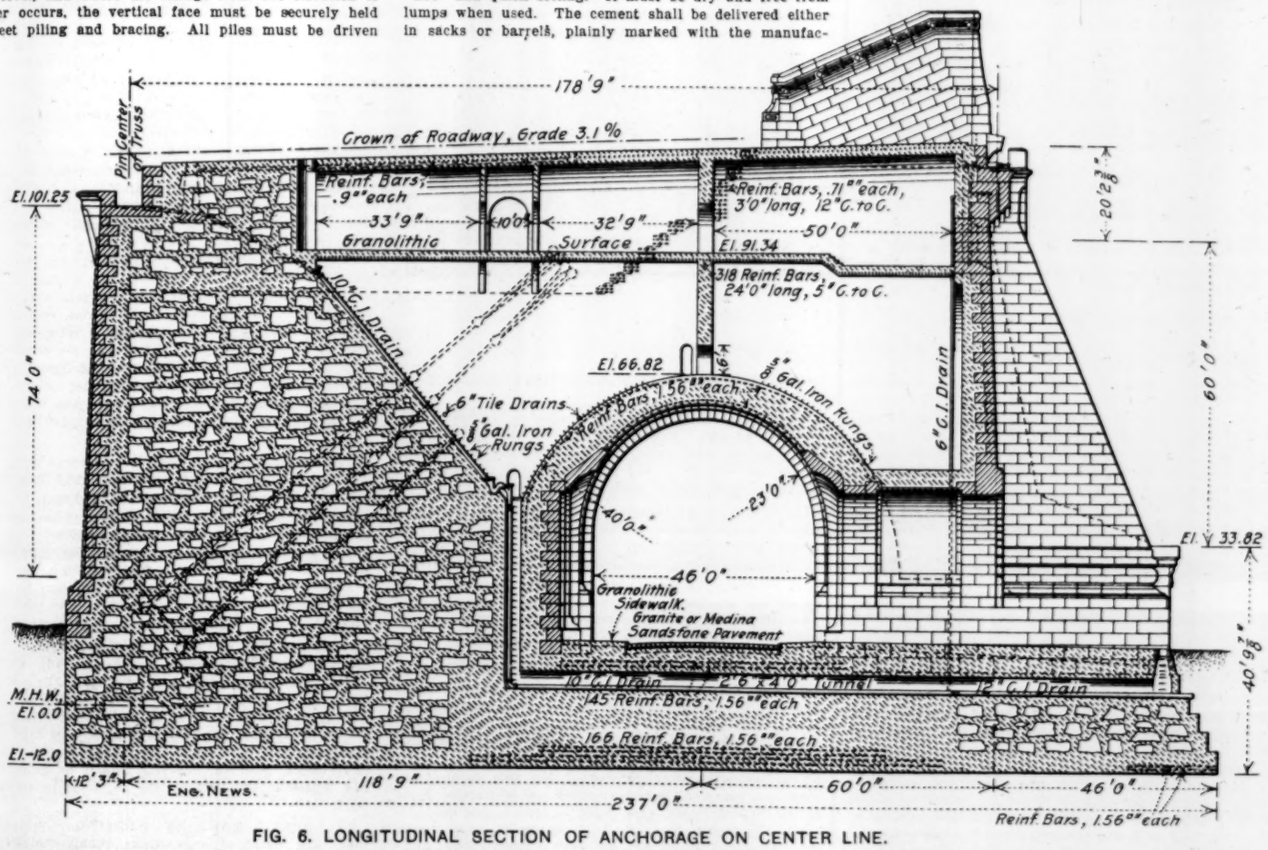


FIG. 6. LONGITUDINAL SECTION OF ANCHORAGE ON CENTER LINE.

the air and water to be as nearly 70° F. as practicable. If the mortar is to contain sand, the sand and cement will be first thoroughly mixed dry. The dry material shall be placed on a glass plate, formed into a crater, and the entire amount of water added at once; the dry material shall then be turned toward the center of the crater so as to absorb the water. After the water has been absorbed, the mixing shall be continued 1½ minutes. The mortar, when ready for the mold, shall be of such consistency that a small quantity taken in the hand and patted into a round form, until water flushes to the surface, shall not stick to the hand, and when allowed to fall from a height of 18 ins. the ball shall retain its rounded form without showing any cracks.

The briquettes shall be of standard form, having a minimum section at the middle of 1 sq. in. The mortar shall be worked into the mold thoroughly by thumb pressure and finished on both sides with a trowel. The briquettes shall be left in air under a damp cloth or in a moist closet for 24 hours and then immersed in water maintained at a temperature of about 70° F. They shall remain in the water until immediately before being broken. In the tension tests the stress shall be applied at the rate of 400 lbs. per minute.

The tensile strength of any carload or 40-ton lot of cement shall show not less than the following values:

	7 Days. 1 day in air, 6 days in water, lbs. per sq. in.	28 Days. 1 day in air, 27 days in water, lbs per sq. in.
Neat cement	400	600 and also average 15 % greater than at 7 days.
Cement 1—3 sand	150	240 and also average 25% greater than at 7 days.

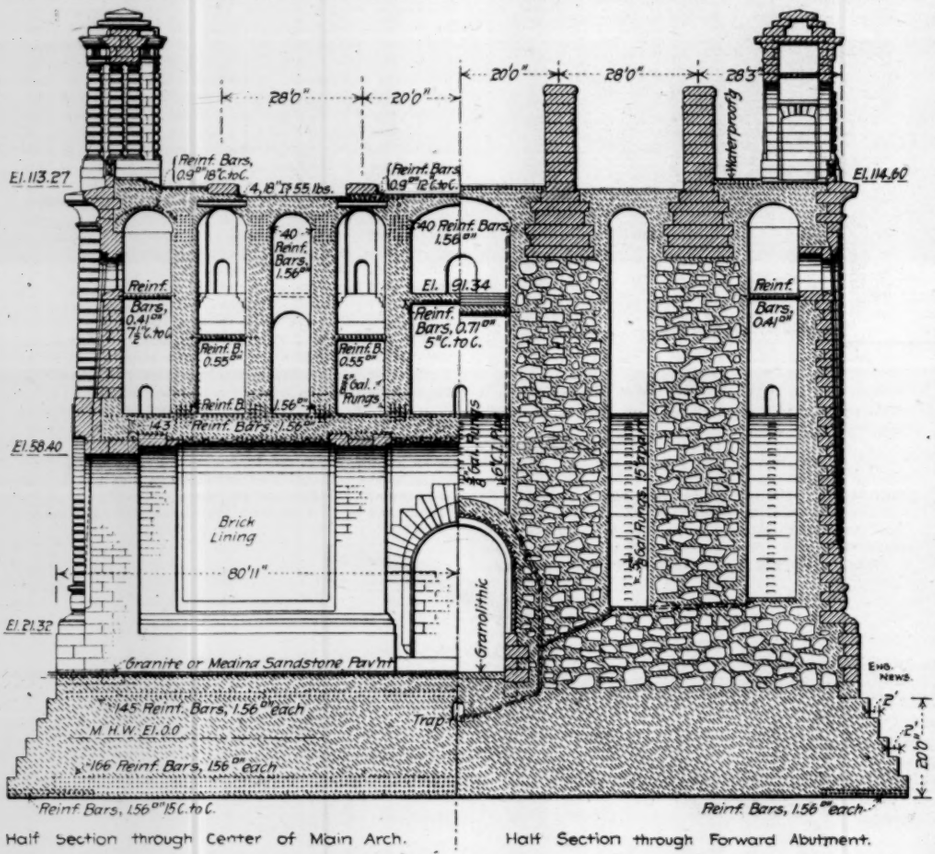
In case any lot of cement fails in one or more of the above requirements, it shall be given a second test for such requirement or requirements, which test shall be final and binding.

The engineer may direct that tests be made, from time to time, of tensile strength and soundness, extending over longer periods than 28 days; if cement so tested shows a reduction of strength with increased age, or at any time fails in respect to soundness, the engineer may prohibit the future use of that brand of cement and require that another brand be substituted. The laboratory tests given above are not final. Should the engineer at any time claim any lot of cement damaged or questionable in any respect, the same shall be rejected, although it may previously have met other tests.

SAND.—Sand, wherever used, shall be coarse, clean, sharp and silicious, not containing more than 0.5% of mica, dirt or clay, or of all combined, and equal in quality to the best Cow Bay sand. If required by the engineer, it shall be screened.

BROKEN STONE.—All broken stone to be used for concrete must be strong, tough and hard, breaking in cubical forms with angular fracture. It must be machine broken and consist of the entire output of the crusher, except the dust, which, if required by the engineer, must be screened off. The stone must vary in size, so as to give the least possible voids, and free from any substance liable to disintegrate or impair its strength, and must be clean and free from any considerable proportion of spalls or flat pieces. Stone for No. 1 concrete shall be of granite, trap rock or limestone, of a size not to exceed 1¼ ins. largest diameter. Stone for No. 2 concrete shall be of granite or trap rock not to exceed ¼-in. largest diameter.

WATER.—All water used for concrete, grout or mortar, and for washing or wetting stone or brick, must be fresh and clean.



Half Section through Center of Main Arch. Half Section through Forward Abutment.

FIG. 7. TRANSVERSE SECTIONS OF ANCHORAGE.

GRANITE.—All granite must be equal to the best grade of Maine granite. It must be fine and uniform in grain and color, perfectly sound and free from sap, seams, dries, cracks and defects of any kind calculated to impair its strength, durability or appearance.

LIMESTONE.—All limestone must be the best quality of gray limestone, capable of standing without failure a pressure of at least 12,000 lbs. per sq. in. It must be perfectly sound and free from sap, seams, dries, cracks or defects of any kind calculated to impair its strength or durability.

SANDSTONE.—Sandstone must be in strength, quality and freedom from defects fully equal to the limestone specified above.

STEEL.—The city will furnish to the contractor f. o. b. lighter at the foot of Pike Slip, all the structural steel, iron and bronze work shown on the plans and referred to in the contract, except the reinforcing bars, dowels, cramps, ladder rungs and other metal work, as hereinafter specified.

The contractor shall furnish at his own expense, as shown on plans and herein specified, all reinforcing bars, dowels, cramps, water pipe, manhole covers, gratings, etc.

All reinforcing bars shall be made of open-hearth steel, especially rolled for this work. The steel for reinforcing bars shall have an ultimate strength of 60,000 to 68,000 lbs. per sq. in. and an elastic limit of not less than 33,000 lbs. per sq. in. It shall bend cold without fracture around a bar of diameter twice the minimum thickness of the bar. Reinforcing bars must be straight and free from flaws in any degree calculated to impair their strength. They must be of such form as to give the most thorough bonding with the concrete and be a[pproved] by the engineer.

Dowels shall be made of the best quality of me[table] steel, and must bend without fracture aroun[d bar] of diameter twice the thickness of the bar. Cramp[s all] be made of rolled steel forged, or cast steel. If [of] rolled steel, this must be equal in quality to that [for] reinforcing bars. If cast steel, this must show [a] mate strength of at least 65,000 lbs. per sq. in. [an] elastic limit of at least half the ultimate. It mu[st] without fracture around a bar three times the thick[ness] of the test piece. Whether forgings or castings, the [pieces] must be sound, free from flaws, straight and accur[ate] to dimensions called for.

All iron castings shall be of the best quality of [gray] iron, sound and true.

CONCRETE.—Concrete shall be of two classes, [No. 1] and No. 2. No. 1 concrete shall be made of one pa[rt] ment to seven parts sand and 1½-in. broken stone. [The] ratio of sand to stone shall be about one to two, the [exact] ratio to be determined from time to time by the [engi]neer, depending upon the character of material used.

No. 1 concrete shall be used in foundations, in join[ts of] Cyclopean masonry, in concrete walls and arches a[s in] backing, except as hereinafter otherwise specified. [No.] 2 concrete shall be made of one part cement to six [parts] sand and ¼-in. broken stone. The ratio of sand to s[tone] will be about one to two, the exact ratio to be determ[ined] from time to time by the engineer, depending upon the character of material used. No. 2 concrete shall be [used] about steel wherever it is embedded in concrete (ex[cept] in case of reinforcing bars), in the concrete-steel be[ams] and the concrete-steel slabs forming the lower floor.

Concrete shall as a rule be machine mixed. The [com]ponent materials in the concrete must be accurately me[as]ured. The mixers must be of a design approved by the engineer, and be so operated as to produce thoroughly mixed and uniform concrete. Concrete mixed by hand can be used only by special permission of the engineer. When so mixed, sand and cement must first be mixed dry, and then the proper quantity of water added and the whole thoroughly worked into plastic mortar. Then the broken stone, previously wet, shall be added, and the whole turned until completely mixed. Concrete must be finally placed within forty minutes after mixing. In measuring materials for concrete, one volume of cement shall be taken as four bags of 95 lbs. net weight each, or one barrel of 380 lbs. net weight. One volume of sand or broken stone shall be taken as 3½ cu. ft. shaken down.

Concrete shall as a rule be deposited in layers from 8 to 10 ins. thick, and shall be thoroughly rammed until it forms a compact, uniform mass, free from voids. Twelve-pound metal rammers shall as a rule be used, and such other rammers or pneumatic machines as may be required by the engineer to secure thorough work in difficult places. After ramming, surface of concrete shall be kept wet until it fully sets. Concrete shall be laid and rammed in layers, as far as praticable, normal to the direction of the pressure it carries. When a course of fresh concrete is to be laid on a surface of concrete already set, the surface shall be thoroughly cleaned and washed, so as to make a firm bond between the two layers. If required by the engineer, the surface of the set concrete shall be roughened with a pick and a bed of mortar ½-in. thick spread before laying the fresh concrete. A grout of neat cement may be substituted for the mortar, at the option of the engineer. No cement in any form shall be used in freezing weather, except by special authority from the engineer and under conditions prescribed by him. Mortar, grout or concrete which has begun to set shall not be used; retempering will not be allowed.

MORTAR.—In preparing mortar for stone masonry, the specified amounts of cement and sand shall first be mixed dry to a uniform color. The water shall then be added in such a manner as not to wash out any of the cement, and the mixing proceeded with until the mortar is thoroughly mixed and of uniform consistency. The proportions of cement and sand will be 1 to 2½ by volume unless otherwise specified. Consistency of mortar shall be fixed by the engineer.

GROUT.—Grout will be made in the proportion of one part of cement to one part of sand by volume. The materials shall be thoroughly mixed dry, and water then added, while the mixing proceeds, until the grout is of the required consistency. The mixing shall be continued vigorously until the entire amount mixed is used, so as to prevent the separation of sand and cement. Grout shall be used about the imbedded steel, for filling small voids, wherever concrete cannot be properly used, and wherever required by the engineer to insure tight work.

CYCLOPEAN MASONRY BACKING.—Cyclopean masonry will consist of large stones roughly cubical in form with approximately horizontal beds and vertical joints, bedded in and filled around with concrete. It will form the backing of bases, buttresses and filling of rear of anchorage, as shown on plans. All stones shall be granite, limestone or sandstone, of quality hereinbefore specified, or other stone in the opinion of the engineer equal to the best quality of limestone. Stones are not to be dressed

t must be either quarried in roughly cubical form, or e so shaped by knocking off protruding edges or cor- s. At least 75% of the large stones used must not be s than ½ cu. yd. in volume. In no case must the larg- horizontal dimension of a stone be more than twice maximum depth. Large stones must vary in size, so to permit of bonding, as hereinafter prescribed.

mall stones not less than ¼ cu. ft. in volume may be d in the joints between large stones where these joints more than 18 ins. wide. Large stones must be laid prescribed by the engineer, in a manner to break up as as practicable continuous horizontal beds. They all be so laid as to be not nearer to each other at any int than 6 ins. They must be laid in a bed of concrete less than 6 ins. thick, being dropped or worked down th bars in such a manner as to insure thorough bed- ding. Should the inspector doubt the thorough bedding of any stone, it must be raised and reset in a newly prepared bed.

No. 1 concrete shall be used for beds and joints of cyclopean masonry. All joints less than 18 ins. wide shall be filled entirely with concrete, thoroughly rammed. Joints more than 18 ins. wide may be filled partly with small stones, thoroughly bedded in concrete. Such stones must not be nearer each other or the larger stones than 3 ins., and must be deposited in layers upon a bed of concrete not less than 4 ins. thick. Then the stones must be entirely covered with concrete thoroughly rammed, and deposited to a thickness not less than 4 ins. over the tops of the stones. Then another layer of stones will be laid, rammed and buried in concrete in the same manner, until the joint is filled. All stones must be thoroughly cleaned and wet before laying. No stone in the cyclopean masonry shall be nearer any face of the masonry than 1 ft. Concrete faces of cyclopean masonry shall be built to accurate forms, and shall be smooth and uniform.

GRANITE FACING.—The anchorage shall be faced throughout with granite, coursed and bonded as shown on plans. Headers shall be in no case less than 4 ft. deep and stretchers in no case less than 2 ft. deep. In the molded courses, quoins, or other places where special bonding is required to make the work secure, the stones shall be of such greater depth as may be deemed necessary for this purpose by the engineer. Wherever on the plans the face stones are shown of equal length, in general every third stone shall be a header. No dog holes for handling will be allowed in any exposed faces of masonry.

The side walls of the anchorage over the main archway shall be composed throughout of granite ashlar, with ½-in. joints and with the inner faces cut to approximately true planes. All stones of the molded course next below the coping shall be cramped together with horizontal cramps, there being one cramp to each joint. The coping course shall be doweled to the molded course with two ⅞-in. dowels to each stone. The three upper courses of the buttresses and all stones of the cable pedestals having horizontal joints shall be securely cramped or doweled. Cramps and dowels shall be used at other points where the corbeling of courses is enough, in the opinion of the engineer, to require them. Dowels and cramps must be bedded in mortar. Dowels shall be ⅞-in. in diameter, and extend 6 ins. into each stone.

PEDESTALS, BASES AND BUTTRESSES.—All stones in the granite pedestals under the cable saddles shall be cut throughout to exact dimensions with ½-in. beds and joints. Beds and joints shall be fine pointed throughout, and true to straight edge. No cavities will be allowed more than ½-in. deep, or covering in the aggregate more than 10% of the surface of any bed or joint. All exposed edges shall be exact to plan and free from flaws.

The coping stones of the pedestals shall have their tops rough pointed before setting, being about 1 in. full. After setting, the tops shall be dressed and rubbed down, so that no part of it shall vary from the required plane by more than 1-64-in. when tested with an accurate steel straight edge. The pedestal stones laid on an incline must be set with great care to insure all beds being perfectly filled with mortar.

All stones must be trial bedded in a manner prescribed by the engineer, and shall be lifted at least once or as many times as may be required by the inspector to insure thorough bedding. Should it be required by the engineer, beds under the stones laid on an incline shall be rammed full of soft mortar after the stones are accurately set on wedges. Joints shall be filled with mortar thoroughly rammed in with thin metal blades. Before setting, all stones shall be washed perfectly clean, and be wet when set, and stones already laid must be cleaned and wet to receive the mortar bed for the next stone above.

Mortar for all pedestal work shall be mixed in the proportion of one of cement to two of sand. The lower courses of the pedestals will be stepped and bonded into the concrete of the bases and buttresses as shown on plans. Beds and faces of stones bonding in with the concrete may be rough. All pedestal stones below the roadway and not exposed need not be uniform in color, but must in all other respects be of the same quality as those in the upper courses of the pedestals. The concrete about the lower courses of the pedestals and between the bottoms of the pedestals and top courses of the cyclopean masonry of the bases and buttresses shall be No. 1 concrete. That portion of the concrete under the pedestals shall be built to forms so as to receive the pedestals coursed and bonded as shown in plans. The concrete under the pedestals shall be rammed as nearly as practicable in layers parallel to the beds of the pedestals. The bases and backing of buttresses shall consist of cyclopean masonry with interior faces built to accurate forms.

The work described is being performed under the direction of Mr. Geo. E. Best, Commissioner of Bridges; Mr. O. F. Nichols, M. Am. Soc. C. E., Chief Engineer; Mr. R. S. Buck, M. Am. Soc. C. E., Consulting Engineer; Mr. H. D. Robinson, Engineer-in-Charge, and Carrère & Hastings, Architects.

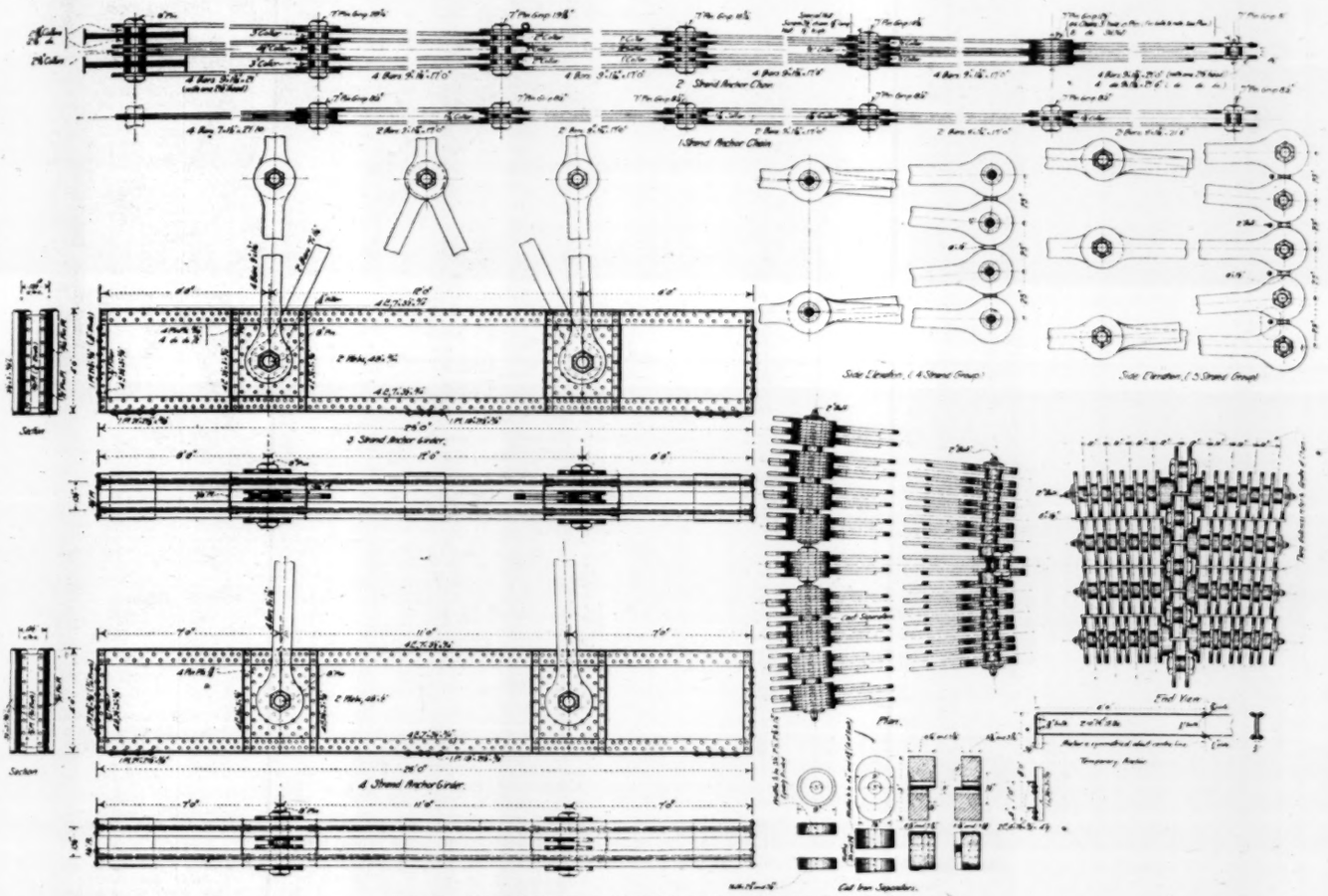


FIG. 5. DETAILS OF ANCHORS AND ANCHOR CHAINS.

REPORT ON FILTERING THE CROTON WATER SUPPLY, NEW YORK CITY.

The filtration of the Croton section of the New York water supply was included in the recommendations of the Burr-Hering-Freeman Commission in their report of a year ago. Some weeks back the subject was again brought to the front by Mr. J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer of the Croton Aqueduct Commission, who recommended that work on the easterly half of the Jerome Park reservoir be discontinued, since if filtration were to be adopted the reservoir would need to be covered, and since covering could be more cheaply done during than after the original construction. Some of the questions involved were referred to Messrs. Allen Hazen, M. Am. Soc. C. E., and Geo. Whipple, Assoc. M. Am. Soc. C. E., of New York. On Dec. 10 these engineers submitted a report to the Aqueduct Commission in which they concurred in Mr. Smith's recommendation for suspending work on the easterly half of the Jerome Park reservoir, endorsed the general proposals for filtering the Cro-

ton supply and discussed various phases of the filtration problem.

Messrs. Hazen and Whipple state that "there is no question that the Croton water will have to be filtered." The population on the drainage area is increasing and the city cannot limit the increase. The most that the city can do is to acquire property which is or may be used to pollute the water or enforce certain sanitary regulations.

been greatly advanced in the last ten years, and it is likely to be still further advanced. Ten years ago, measured by the standards then current, Croton water was very good water. To-day there is considerable doubt as to whether it can still be called pure and wholesome. Purification must be resorted to at an early date if the water is to be maintained reasonably pure and wholesome, and equal in quality to the supplies of other cities.

As to the cost of filtration, Messrs. Hazen and Whipple suggest that a higher rate could be employed for slow sand filters than was suggested

event the uncovered portions would be upon in case of emergencies only. A deep, tunnel from Jerome Park reservoir to Park would equalize fluctuations in co tion, thus lessening the need for filtered storage in Central Park. Such a tunnel, ently, would cost no more than covering th reservoir in Central Park and, aside from i tion to filtration, would be of value in

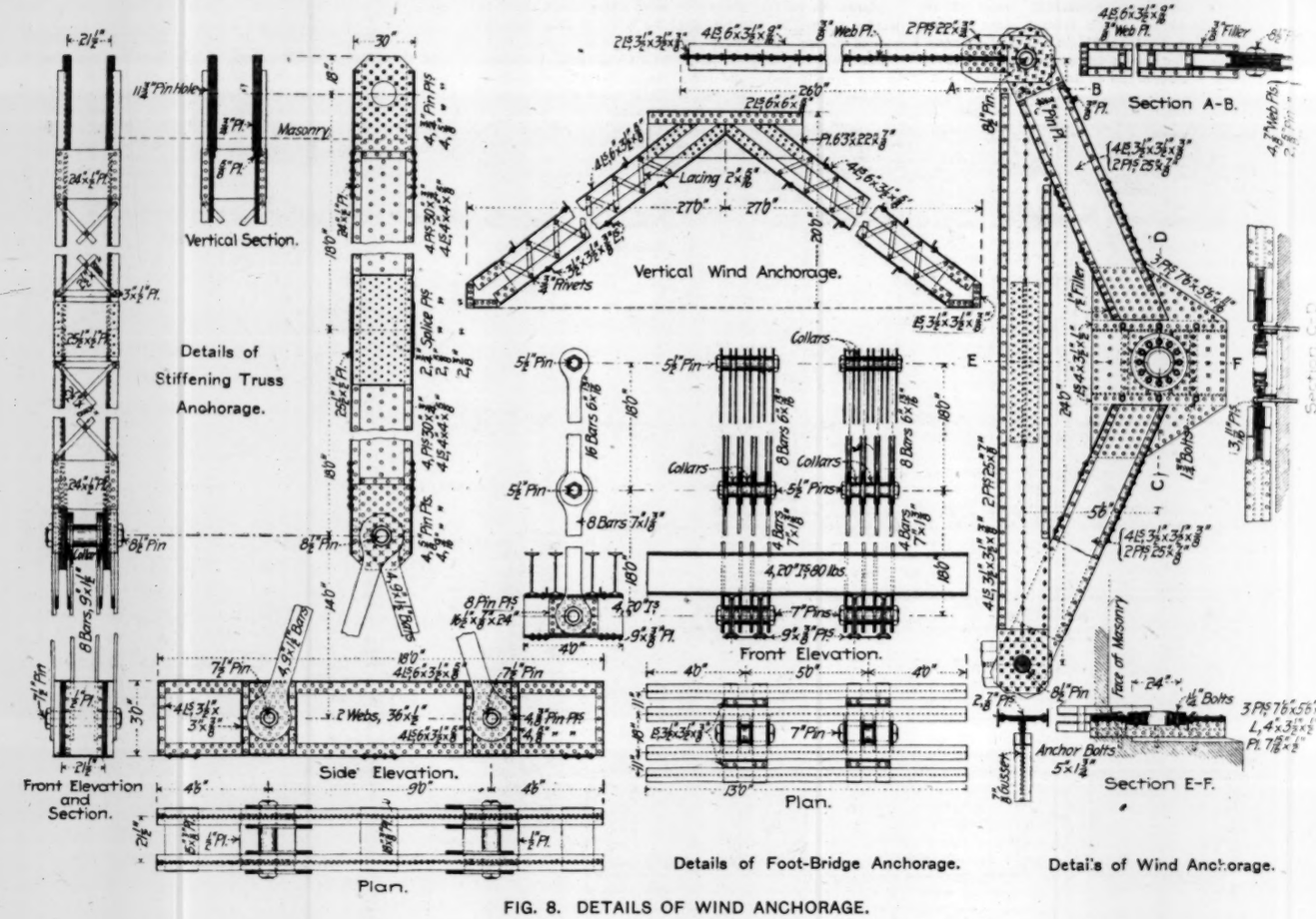


FIG. 8. DETAILS OF WIND ANCHORAGE.

Although helpful, neither of these means, within reasonable financial limits, are sufficient.

Filtration is required to reduce slight yellow discoloration, turbidity and the occasional tastes and odors to which the water is subject. Compared with western and southern streams the turbidity is low but it accumulates in pipes and fixtures and on being flushed out detracts seriously from the attractiveness of the water. The organic growths which give rise to taste and odors take place chiefly in the higher reservoirs in the drainage area and in the Central Park reservoirs. The reservoir formed by the new Croton dam "will tend to improve the water from a sanitary standpoint," but will favor the growth of organisms which produce tastes and odors.

It is likely that copper sulphate might be used to prevent organic growths, but this would not reduce turbidity or color, "nor would it materially reduce the sanitary reasons for the filtration of the water." Further:

> To protect the water from a sanitary standpoint by the use of copper would require a continuous and liberal treatment, and would result in the constant presence of a considerable amount of copper in the water supplied to the city, a condition which should not be tolerated.

New York, the report states, is losing its high rank as a city with a water supply superior to most other American cities. Boston and a number of the suburbs of New York are already supplied with water which is better than the Croton, and it is probable that the same will soon be true of Philadelphia and Washington. Continuing the report states:

> The standard of purity for public water supplies has

by the Freeman-Hering-Burr Commission, thus perhaps reducing the commission's estimate from $8,500,000 to $7,000,000 to $6,000,000.

The extra cost of covering the easterly half of the Jerome Park reservoir may be taken at $2,500,000 to $3,000,000; of covering the two old Central Park reservoirs, of 200,000,000 gallons capacity, $1,000,000; of covering the new reservoir in Central Park, holding 1,000,000,000 gallons, $3,500,000. The report suggests that so much storage capacity for filtered water might be quite unnecessary; in fact, that it might be sufficient if the small old reservoirs in Central Park, and possibly only a portion of the easterly half of the Jerome Park reservoir were covered. In that

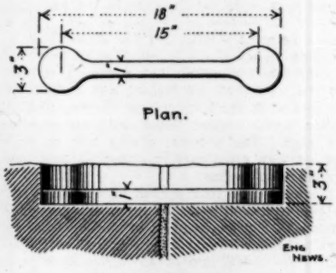


Fig. 9. Cramp for Masonry.

emergencies. As bearing on this general subject, the report says:

> In the routine operation of a water-works system there is no necessity for very large distributing reservoirs. They are of use only for meeting emergencies, and water-works construction and machinery are now, in the best works, so substantial that the emergencies requiring them are far less likely to occur than was the case when the first aqueducts were built.

THE BLACKWELLS ISLAND BRIDGE is to be enlarged to carry six railway tracks instead of four. At a recent meeting the Board of Estimate and Apportionment of New York City authorized the Bridge Commissioner to change the accepted plans for this bridge so as to provide for two additional railway tracks. The changes will increase the cost of the structure about $360,000.

THE SECOND 96-MILE PIPE LINE FOR MANCHESter, England, was opened about Nov. 1. Like the first one, opened in October, 1894, and which this one parallels, the new pipe line has a nominal capacity of 10,000,000 Imp. or 12,000,000 U. S. gallons, and brings water from Lake Thirlmere, in Cumberland. Strictly speaking, these pipe lines are not 96 miles long, since some 14 miles of tunnels and 37 miles of concrete aqueduct were completed to the full capacity of 50,000,000 Imp. or 60,000,000 U. S. gallons at the outset. After a few years of use the capacity of the first line of pipe, which was 40 ins. in diameter, decreased from 10,000,000 Imp. gallons, nominal, to about 9,000,000 Imp. gallons. The second line, therefore, is 44 ins. in diameter, with variations to 40 ins. and to 48 ins. for short lengths. Cast-iron pipe was used on the new contract, except on bridges and under very heavy pressure, where lap-welded steel was substituted. Mr. Geo. H. Hill, M. Inst. C. E., of 3 Victoria St., London, S. W., is engineer for the Manchester extensions. The foregoing information has been taken from London "Water" for Nov. 15, 1904.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering.

Entered at the New York Post-Office as Second-Class Matter.
Published every Thursday
at St. Paul Building, 220 Broadway, New York, by
THE ENGINEERING NEWS PUBLISHING CO.

PUBLICATION OFFICE, 220 BROADWAY, NEW YORK.
CHICAGO OFFICE, 1636 MONADNOCK BLOCK.
BOSTON OFFICE, 170 SUMMER ST.
ATLANTA OFFICE, AUSTELL BUILDING.

FOREIGN AGENTS.

ARCHIBALD CONSTABLE & Co., LTD., 16 James St., Haymarket, S. W., LONDON.
RUDOLF MOSSE, BERLIN and HAMBURG, GERMANY.
JOHN F. JONES, & CIE. 31 Rue de Faubourg Montmartre, PARIS.

SUBSCRIPTION RATES: United States, Canada and Mexico, One Year, $5.00; 6 months, $2.50; 10 weeks, $1.00. To all other countries in the Postal Union: Regular Edition, One Year, $8.00; Thin Paper Edition, One Year, $5.00 (21 shillings). SINGLE COPIES, 15c.

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An unprecedented feature in the first six contracts for the New York Barge Canal, bids for which were received last week, is the publication of the State Engineer's itemized estimates as a part of the specifications. We use the term unprecedented with reference to New York State contract work, but, in fact, we know of no contract of magnitude anywhere in which this method has been followed.

The principal object in making the engineer's estimate a part of the specifications was to prevent unbalanced bids by requiring the contractors to submit no bid on any single item more than 20% in excess of the engineer's estimate on that item. Failure to follow this instruction was apt to cause the rejection of the whole bid, although a bid might be accepted if the contractor could convince the State Engineer and the Advisory Board that in exceeding the 20% limit on any item his bid was not unbalanced. It is noteworthy, however, that very few bidders took the risk of exceeding the 20% limit, although on certain items many of the contractors bid to the allowed limit, indicating, perhaps, that they regarded the State Engineer's estimate as too low. But, considering the multitude of items and the number and ability of the contracting firms that submitted bids, the State Engineer and his assistants are to be congratulated upon the general confirmation of their estimates.

Reference to the detailed statement of the bids given in our Construction News Supplement will show that excavation and concrete were, in most of the contracts, by far the largest items. Naturally the bids on concrete would not be far apart as there are few elements of uncertainty; but it is noteworthy that the bids on excavation are also very close together, with one or two exceptions.

There is no "classification" in the excavation item, rock and earth being lumped together. Had it not been for one thing, this absence of classification would probably have led to higher bidding prices. We refer to the fact that the contractors were furnished with profiles and cross-sections showing in detail the quantity and character of materials that the surveys, soundings and test pits had disclosed. Moreover, at intervals along the line of the canal, samples are kept of all the materials encountered by the boring parties, and these samples are so labeled that a contractor can identify their position by reference numbers on the profiles.

Since there is little or no likelihood that the route or the depth of the canal will be changed from the position shown on the plans, this method of no classification has much to commend it, particularly in view of the political capital that the old Erie Canal classification specifications have furnished.

The no-classification method has its obvious objections, among which are the following: (1) It does not disclose what the contractors consider the earth and the rock to be worth separately; (2) In case of any change of alinement and grade, involving different proportions of rock and earth, there is no unit price in the contract that can be used; (3) In making the monthly estimates the engineer is apparently forced to allow the excavation bidding price regardless of the kind of material excavated, resulting in the payment of an unduly high price for the first work done—the earth stripping.

Offsetting these objections in this particular case, we have: (1) The State Engineer is dependent upon none of his many subordinates for the proper interpretation of a classification clause; (2) there can be no lawsuit arising from the interpretation of a classification clause; and (3) neither the judgment nor the integrity of the engineering department can be assailed in respect to excavation classification.

Our representative found none of the contractors at Albany who complained of the lack of a classification clause, or who seemed to think that bids would be higher in consequence of such lack; for the carefulness with which the subsurface surveys have been conducted appear to have eliminated most of the elements of chance. A few contractors, however, expressed themselves as being in favor of more test pits to check the borings.

Apparently all contractors counted upon working an 8-hour day, as called for in the contract, regardless of the recent court decision which seems to have nullified the 8-hour law, at least in so far as it applies to municipal work. The constitutionality of the law as applied to contract work for the state will probably have to be tested by a suit some time in the future. In view of the many million dollars involved in the Barge Canal contracts, to say nothing of road contracts, still to be let, it is to be hoped that the 8-hour law as applied to state work will be brought up again in some final test case.

The recent experience of Civil Engineer Peter C. Asserson, U. S. N., retired, calls for some comment. This officer was retired from active service with the rank of Rear Admiral and the corresponding pay; but when he used the title of "Rear Admiral" upon his visiting card, a line officer of similar grade protested to the Navy Department that Mr. Asserson, being a staff officer, was unauthorized to use a title that "belonged only to officers of the navy empowered to actually command ships and squadrons." The Secretary of the Navy, by directing Mr. Asserson not to use the title of "Rear Admiral," either in official or personal correspondence, practically supports the contention of the line officer.

The whole question resolves itself into—What privileges are conferred upon a staff officer who has "relative rank" with a line officer? Does this word "relative" mean any more than that the staff officer in question is entitled to receive the same pay as the line officer of a corresponding grade? The Secretary of the Navy evidently thinks that the staff officer *is* not warranted in assuming the line title and he has so ruled. In the military and naval service, rank means authority, precedence, relative importance among other officers and social status. It means much more than the pay in the majority of cases; and it is not at all remarkable that staff officers of high grade should object to the retention of a staff title that means nothing to a public very familiar with the designation of "General" and "Admiral." It is said that an effort is to be made in the next Congress to correct this anomaly by authorizing the staff officer to assume the corresponding line title. But a "Civil Engineer" in the corps of civil engineers, or a "Chief Engineer" in the steam department, never performed the duties of the "Rear Admiral"; though by virtue of appointment to the head of a Bureau either may have been entitled to the pay of a Rear Admiral by reason of equivalent responsibility, or either of these men may be retired with this pay by reason of length of service. At the same time, as measured by the importance of the services performed, both engineers may relatively rank higher than many of the actual Rear Admirals, and this fact should be publicly recognized in title as well as in pay.

Could not the problem be solved by some appropriate affix to the more familiar and more generally honored title—as Engineer-Rear Admiral, Surgeon-Rear Admiral, Pay-Rear Admiral? These designations may seem awkward in a sense, and perhaps would not suit all of the staff officers; but some such affix would have the merit of removing the real difficulties in the present case—which are mainly social, and of placing officers of corresponding length and efficiency of service upon an equal footing before the public. There is already a precedent for such action in the case of the Army, where we now have Surgeon Generals, Paymaster General and Quartermaster General. Some such law authorizing the use on all occasions of a distinctive and sufficiently honorable title would remove much friction.

In view of the approaching examination of candidates for Assistant Civil Engineers in the Navy to be held on Jan. 9 next, as announced in our issue of Nov. 17, it is of interest to record that in the examination held in August last, "none of the few applicants taking the examination passed satisfactorily," to quote from the Report, just issued, of the Chief of the Bureau of Yards and Docks, referring to the approaching examination, the Chief expresses the hope that "because of the longer time allowed for publicity regarding it and for preparation, a larger number will avail of the opportunity and the vacancies now existing be filled."

As vacancies for five Assistant Civil Engineers now exist in the Corps, any engineer able to pass a satisfactory examination and fulfil the other requirements as to age, physical condition, etc., will stand a good chance of obtaining a commission.

Regarding the emoluments, etc., of the position, they were set forth in detail in our issue of Nov. 17; but it may be added that in the report above mentioned Chief Endicott recommends that the pay of Naval Civil Engineers should be increased and made the same in all respects as the pay of Naval Constructors.

HOW CAN EFFLORESCENCE ON CONCRETE BE PREVENTED?

The white incrustation so commonly seen upon the face of concrete walls is so unsightly that it prejudices many architects, and engineers as well, against the use of concrete where appearance is an important consideration. Efflorescence is particularly noticeable just below all horizontal joints in concrete walls, that is just below the line between one day's work and the next. For example, where a section of concrete retaining wall is first built, and a day or more afterward a concrete coping is placed upon the wall, there is almost sure to be an unsightly white stain formed just below the coping. The cause of the stain appears to be this: Where concrete is laid "dry" and rammed, or where it is laid "wet" and spaded, the horizontal surface of the finished concrete layer consists of a mortar richer in cement than the body of the wall. Even if it is not richer at the time the concrete is left to set, it is usually made richer when, a day or so later, a sprinkling of neat cement is placed upon the surface to act

as a binder for the coping. Now, as is well known, a mortar rich in cement is almost impervious to water, and as a consequence when a horizontal stratum of such mortar occurs in a wall, it stops percolating water and diverts it to the surface of the wall. Therefore, as some of the water in a fresh concrete coping settles by gravity, it is intercepted by the impervious surface, and is forced out along the joint between the new and the old work. Since many cements contain sulphates of magnesium and calcium, and since these sulphates are slightly soluble in water, it follows that they will be carried by any water that leaches through the coping and will be deposited in crystalline form where the water evaporates on the face of the wall.

In the course of time, rain water beating upon the face of the wall will dissolve and remove these sulphates particle by particle, but this process is slow at best.

While efflorescences are most pronounced below copings, and wherever horizontal joints occur in concrete, they are not confined to such places, but frequently appear upon the face of a wall that has been built without the horizontal joints due to stoppage of work at night. The use of "wet" or "sloppy" concrete appears to have resulted in greater amounts of efflorescence than was formerly seen when "dry" concrete was in fashion. But regardless of the amount of water in concrete at the time of its deposition, efflorescence more or less pronounced is quite certain to appear after a period of wet weather. For, if the face of the concrete becomes saturated to a depth of a few inches, and if soluble salts exist in the concrete, these salts will ultimately saturate the water. Then the deposition of the salts upon the concrete surface begins, due to the more rapid evaporation of the water there, and due to the lower temperature at the surface. Since solubility decreases with decrease in temperature, and since the force of osmosis causes a flow of the solution from parts of the concrete containing a stronger solution to parts containing a weaker, it follows that the face of a moist wall may be fed with soluble salts until all the salts are leached out of the wall.

These explanations of the phenomenon of efflorescence are necessary in order that methods of prevention or cure may be intelligently considered. To prevent the formation of efflorescence the most effective method would be to use cement entirely free from sulphates and chlorides. It is not at all likely that this will be done on engineering structures, except perhaps in comparatively rare cases. Nevertheless there are cements in the market that cause little or no efflorescence and others that cause much, and when this fact is recognized it will lead to greater care in specifications governing work where appearance "counts." Perhaps it will be sufficient simply to use a face mortar made of a cement free from soluble salts and made so rich as to prevent percolation of water into the body of the wall. But where thin reinforced walls are made of very wet concrete, this method will probably not be used, due to the difficulty of placing a special face mortar.

A method that is sure to be effective, for a time at least, is to use a Sylvester facing mortar or to apply a Sylvester wash of alum and soft soap to the face. "Szerelmey stone liquid," when applied to the face of concrete, is said to make it perfectly impervious. The durability of either of these washes is unknown to us by our own observation, but for the purpose of preventing efflorescence the durability of the wash may not be an important factor, particularly in heavy concrete structures that can be built with "dry" concrete. For, if instead of building horizontal layers of concrete, the layers are built on a slight slope toward the rear of the wall, and if every other layer is well sprinkled with neat cement, all water moving by gravity in the body of the wall will be diverted toward its rear face. Thus, in time, the soluble salts would be leached out, but never appear on the face of the concrete, for during the time of leaching the face would be impervious, due to the wash applied. Indeed, might not this method of inclining the layers toward the rear of the wall be in itself a sufficient protection against leaching under ordinary conditions?

In Gillmore's "Lime, Hydraulic Cements and Mortar," it is stated that if about 10 lbs. of any cheap animal fat is well mixed with 100 lbs. of quick-lime, which is afterward slaked and mixed with a barrel of cement, the formation of efflorescence will be largely reduced, though not entirely prevented.

We have not suggested the mixture of chemicals with the cement for the purpose of producing salts wholly insoluble, for, unless this were done at the factory, it would probably not be adopted as a satisfactory expedient at the site of the work. Nor have we discussed the removal of efflorescences after their formation, which may be accomplished by the use of solvents, by wire brushes, by sand blasts, by pneumatic tool dressing, and the like; for each of these methods involves waiting, it may be a year or more, until the formation of the efflorescence has ceased. Indeed even after years of waiting, there is no assurance that efflorescence will not appear on a wall, particularly in a city. An exceedingly wet season saturating a wall with water containing sulphuric acid resulting from the combustion of sulphur in coal may be the cause of the formation of sulphates upon the face of a wall many years old. At any rate this has happened in the case of old brick walls.

In view of the growing use of concrete for ornamental structures the subject of efflorescence and its prevention is well worthy of a more careful experimental investigation than it has received in the past. A systematic study of the different Portland cements, and the slag cements as well, would soon show what cements, if any, are wholly free from efflorescence under all conditions. Experiments with facing mortars and with washes would show the best means of protecting concrete walls from the disfiguring streaks of white so commonly seen. There are many subjects besides strength tests that may with profit be investigated in cement laboratories, and not the least important of these subjects is efflorescence.

LETTERS TO THE EDITOR.

Why Are Old Government Section Corners Found Out of Place?

Sir: No doubt all surveyors who have had to "prove up" or re-establish old government corners, find, as I do, almost always a disagreement between the actual and "returned" bearings and distances from the corners to the witness trees, or their stumps. This is laid to the carelessness of the U. S. deputies who set the stakes.

Dorr, in his "Surveyor's Guide," says some surveyors think the deputies used the magnetic meridian when taking the bearing of the witness trees, instead of the true meridian by which their lines were run; but he doesn't say what they think caused the discrepancy in distances. He states that, for his own part, if he can get five out of the eight measurements to agree at a section corner he is satisfied, and calls the rest "mistakes." And that is about as well as any one can do with corners 70 to 80 years old, as they mostly are in my neighborhood.

While the old deputies were careless, I think they set their corners truer than we find them to-day, for I believe there is a constant movement going on in the soil near trees. The live trees around a section corner, while their roots and bases expand, thrust against the surrounding earth and move it imperceptibly but surely, and with it any stake driven into it. A large tree will move a smaller one, and a vigorous tree one of slower growth. If all the witnesses are about equi-distant from the corner the thrust would be balanced; otherwise the movement would be toward the farther trees. If some of the witness trees are cut down those standing will have less opposition and push the more. Thus, in course of time, not only is the stake moved, but also often the witnesses.

That is the reason why we so seldom see a government post otherwise than leaning, though it was set plumb at first. The same happens in a nursery when young trees are staked—their expanding bases soon make the stakes lean over. A row of trees planted near a straight fence will soon throw it out of line, and roots of trees, bulging sidewalks and stone walls are common occurrences.

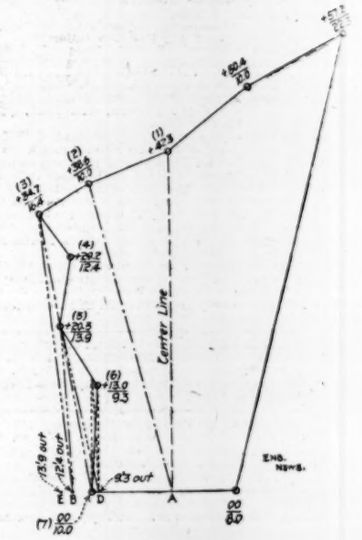
This small-scale land-slide is assisted by dry seasons and wet seasons. In the former the soil shrinks together and when again saturated it swells and rises. In its semi-liquid state it yields easily to the pressure of roots. The corner stake is thus exposed to a perpendicular movement of the soil as well as a horizontal; in dry weather the earth shrinks from it and it becomes loose; then the saturated soil partly floats it, partly pushes it up, until it sometimes falls over and is hidden by vegetation or burned up. The force of the soil movements depend, of course, much upon the kind of soil, as well as drainage.

I should very much like to know the opinion of other surveyors upon this subject. H. E. Lageson.

Starke, Fla., Dec. 10, 1904.

A Method of Calculating the Area of Cross-Sections.

Sir: Through having quite a number of irregular cross-sections to figure, I have developed the following method, having as a basis a similar system used for ordinary cross-sections.



Demonstration.

(a) = 2 × area triangle 1 A 2 + Quantity.
(b) = 2 × area triangles 27 A and 273 + Quantity.
(c) = 2 × area triangle 7 B 3 + Quantity.
(d) = 2 × area triangle B 43 — triangle B 45 = B 543 — Quantity.
Triangle 7 C 6 = Triangle 7 C D.
(e) = 2 × area triangle E 5 D — tri. E 5 B = B 5 D — Quantity.
(f) = 2 × area triangle D 65 — tri. D 6 C = D C 5 — Quantity.

The sketch attached represents a cross-section of an excavation, the area of which is found as follows:

(a) Multiply center cut (1) by the distance out of (2).

(b) Multiply cut of (2) by distance of (3) minus distance of (1).

(c) Multiply cut of (3) by distance of (4) minus distance of (2).

(d) Multiply cut of (4) by distance of (5) minus distance of (3). This is a negative quantity and should be so recorded.

Proceed in same manner, observing the signs, plus or minus, until cut of (6) or the last on left of center line has been used.

Do likewise with readings on right of center line.

The algebraic sum of these will be twice the area of the figure.

Complete Calculation.

			—	+
Left:				
(a)	42.3 × 10.0			423.00
(b)	38.6 × (16.4 — 0.0) = 38.6 × 16.4			633.04
(c)	34.7 × (12.4 — 10.0) = 34.7 × 2.4			83.28
(d)	29.2 × (13.9 — 16.4) = 29.2 × — 2.5		73.00	
(e)	20.5 × (9.3 — 12.4) = 20.5 × — 3.1		63.55	
(f)	13.0 × (10.0 — 13.9) = 13.0 × — 3.9		50.70	
Right:				
(g)	42.3 × 10.0			423.00
(h)	50.4 × (22.3 — 0.0) = 50.4 × 22.3			1,123.92
(i)	57.2 × (8.0 — 10.0) = 57.2 × — 2.0		114.40	
			301.65	2,686.24
				301.65
		Double area		2,384.59
		Area		1,192.295

Operations (a) and (g) are readily combined.

This scheme has the advantages, I think, of simplicity and economy. Whether or not this system is used by others I would like to know. Yours truly,

V. A. Kauffman.

717 Majestic Bldg., Denver, Colo., Nov. 23, 1904.

GAS ENGINE POWER AND SINGLE PHASE TRACTION are combined in an electric railway system being built to operate between Warren, Pa., and Jamestown, N. Y. The power station is being equipped with two 500-HP. gas engines, of horizontal single-crank double-acting type, direct-connected to two single-phase alternators supplying high voltage current for direct transmission (without raising transformers). A 55-HP. gas engine is also provided for the exciter and the air compressor. The engines will operate on natural gas of 1,000 B. T. U. per cu. ft. The cars will be equipped each with four 50-HP. single-phase motors, so arranged that they can operate on direct current within city limits at the terminal. The entire equipment of the line is being supplied by the Westinghouse Electric & Mfg. Co., of Pittsburg, Pa.

AN ERUPTION IN A TEXAS OIL FIELD.

A new oil producing territory is being developed near Humble, Tex., 17 miles north of Houston, and on Dec. 13 the tapping of a gas-charged stratum by the drillers in this field caused unusual phenomena, accounts of which were telegraphed to the daily papers in various parts of the country and gave the impression that a real reservoir of volcanic action had been released by the drillers. The New York "Times" of Dec. 14 had an account headed "Texas Oil District in Violent Eruption," and reading in part as follows:

With a roar that could be heard ten miles away, subterranean forces in the newly discovered oil fields at Humble, Texas, burst through the earth's crust to-day sending clouds of vapor and great stones thousands of feet into the air.

The deep wells to-night resemble miniature volcanoes, the phosphorescent vapors arising from their mouths giving to the entire district the appearance of smoldering fire. Great fissures opened in the earth, and the eruptions of stone and mud lava covered a wide territory.

Every one working in the oil fields fled at the first eruption, and it is well they did, for no life could have withstood the rain of stones that poured down after the fissures opened and the wells began spouting.

Pipes a thousand feet in the earth were hurled into the air, and great showers of rock and mud followed them. All around the wells the earth opened in great gaps, and mud ran from them in streams. In places the earth puffed up into mounds.

This demonstration continued for fifty minutes. Huge rocks were hurled so high in the air they were lost to sight, and mud and rock fell more than a mile away from the oil fields. The damage is at least $30,000, and it is feared to-night that the wells and perhaps the entire field are ruined.

From the Houston "Post" of Dec. 14, it would appear that the above account was the result of some reporter's interview with a badly scared worker returning to Houston from the oil fields. The phenomena observed were unusual enough to be well worth record; but fell far short of the volcanic action so luridly described above. The account given in the "Post" is substantially as follows:

There was a demonstration in the Humble oil field yesterday morning, the like of which has never transpired in any other oil field.

The demonstration began about 9.30 in the morning. Men were at work about the Moonshine No. 3, which blew out the day before. All of a sudden the sand and water began to spurt up around the outside of the 6-in. casing of Moonshine No. 1. Moonshine No. 3 began to throw water and the Landslide well a few feet to the south of Moonshine No. 1 became affected and began throwing sand. The force increased every moment and water began to spurt over the rotary of the Crosbie.

Within a few minutes the sand had burst the floor of Moonshine No. 1, throwing it to one side. A hole was excavated about the opening at the casing and this made the derrick lean over toward the Landslide well and the sand from both wells became heaped all around both the derricks.

There was also a big opening about the hole of the Landslide, and this derrick began to lean. It appeared as though these derricks would topple over on the Davy Crosbie, and the men began placing ropes about their machinery and pulling it out of the way.

While this was going on some of the crowd took panic and began to scatter in all directions, and those running toward the Granberry-Staiti encountered the sand and water blows coming through the small openings and crawfish holes. The water well at the Granberry-Staiti boilers began to boil up and the abandoned Beatty well began to spurt sulphur water high up in the derrick.

The demonstration continued for two or three hours, and then began to subside, and shortly after dinner everything became quiet.

After the panic subsided it was found that the damage was not so great after all. It was found that the only wells really affected were the Monshine No. 1 and the Landslide. The machinery of the latter was greatly disarranged and the derrick was found to be out of plumb. The condition of the hole has not been ascertained as yet, but it is not believed to be damaged to any great extent.

MOONSHINE STILL PRODUCING.

When the reports first came in it was stated that the Moonshine casings had been torn loose and that oil was gushing up around the top of the pipe and becoming mixed with the sand and water and that the whole face of the earth was saturated with oil. This proved to be not a fact. The hole of the Moonshine was not injured a particle. Air was put on in the afternoon and the oil pumped out as on the days preceding.

A gentleman who has been keeping in close touch with the Humble field gave his opinion as to the cause of the demonstration, which is as follows:

"The Higgins No. 2, which blew out with such tremendous force the day preceding, was the cause. There is a 10-in. casing down about 500 ft. in this well. On the former occasion of the blowout there was a 6-in. casing down 800 ft. or to the bottom of the hole, and this is where the gas came from. So long as the 6-in. was in the hole the blow came straight up through the pipe. This was the condition at the time of the first blowout of this well.

"The second blowout began after the 6-in. was taken from the hole. That was the blow of Monday. All the force of gas from the 800-ft. stratum was compressed into this 10-in. hole and shale and gumbo and sand were forced up through it with tremendous force. The 10-in. became clogged up during Monday night. The pressure from the lower stratum continued. The water sand stratum is about 500 ft. from the surface, or near the end of the 10-in. casing in this well. When this casing became clogged the gas began to permeate the water sand stratum, pushing the water with such great force that it sought an opening. The easiest opening was the place where the sand stratum was punctured in the greatest number of places, and here the water and sand were forced, finding their easiest opening at Moonshine No. 1, where it shot up about the outside of the casing, and then the holes near this well became affected."

That this theory is a correct one is demonstrated by the fact that the Mitchell, which is cased off down below the sand stratum, was not affected. The condition of Moonshine No. 1, which was found to be uninjured, is also a demonstration that the theory is a correct one, showing that the pressure did not come from the lower end of this hole.

At the hotel yesterday afternon as soon as the reports began coming in there was a great deal of excitement, and the remark was frequently heard that hades had turned all her batteries on Humble in the effort to break through to the surface of the earth. There were many rumors afloat to the effect that the field was about to be abandoned.

When the more conservative oil men began arriving at the hotel in the evening there was quite a change and the panic began to subside.

Those who had machinery in the field who made any statements at all were emphatic in saying that they intended to keep their holes going down until they punctured the oil stratum and brought in the biggest oil field that was ever discovered.

PLANS FOR THE PANAMA CANAL WORK.

As many of our readers will remember, the House Committee on Interstate and Foreign Commerce visited Panama this fall and viewed the work in progress on the Isthmian Canal. During its visit a statement concerning the progress of the work and the plans under consideration was made to the Committee by Mr. John F. Wallace, Chief Engineer. The text of this statement was given by Chairman Hepburn of the Committee to the Associated Press, and was made public in the daily newspapers served by it on Monday, Dec. 19. We print this statement, as published in the "New York Tribune," as follows:

The isthmus is transversed by a mountain range, the summit of which is approximately 12 miles from the Pacific and 35 miles from the Caribbean. Originally a gorge evidently existed from the Caribbean near Colon to near Gamboa, and extended beyond that point in an easterly direction, forming the upper basin of the Chagres River. After filling this gorge with an alluvial deposit, the Chagres has swung itself from one side of the valley to the other. The result is that the thread of the original gorge cannot be followed or found from surface indications, and it is only by drilling to bedrock that exact information in regard to any particular locality can be obtained. The presence of bowlders in this alluvial deposit also explains the reason why engineers not taking time to go into the rock far enough to determine its actual character have been misled into thinking they had struck bedrock. To determine the most feasible plan for the construction of the canal will require a most careful and comprehensive examination not only of surface conditions, but of the sub-surface.

After following the valley of the Chagres to Gamboa, the line of the canal follows a tributary, called the Obispo, up to the summit of Culebra, and thence down the valley of the Rio Grande into the Bay of Panama. The summit at Culebra was originally about 300 ft. above the sea level, and is the lowest point in the divide along the entire length of the Isthmus of Panama. The plan of the former commission provided for a dam of practically 100 ft. in height, above sea level, at Bohio, with a water level of 90 ft. above sea level. This place was selected on account of the fact that at that point the hills on either side of the Chagres come comparatively close together, being about 1,500 ft. apart and from the surface indications it seemed a favorable place for the construction of a dam. But the indications are that this locality will be an unfavorable and expensive one for the construction of a high dam.

The instructions to the chief engineer by the present commission were to make full and thorough examinations of the canal route, the manner of doing the work and the various plans which might suggest themselves, and to lay before the commission the result of these examinations and his final recommendation.

The first plan to be considered, the one estimated upon by the former commission, is the possibility and practicability of a high dam, or proper foundation for a high dam at Bohio, on which depends the advisability of constructing a high level canal, with the surface of the water 90 ft. above sea level.

The second plan under consideration is a summit level of 60 ft. above sea level. Constructing a canal on this plan admits of two different methods of treatment. First, the construction of a dam 60 ft. above sea level, at Bohio, with two locks of 30 ft., there being two locks on the western slope; second, the construction of a dam 60 ft. above sea level, at Gatun, eight miles from Colon, with two 30-ft. locks in the same neighborhood. The adoption of a 60-ft. level also will render it necessary to construct a dam at Gamboa, in order to provide a reservoir to accumulate water enough in the wet season to furnish water for the summit level of the canal. The construction of a dam at Gamboa, in connection with this, would also control the Chagres River, except that it would be necessary to provide a safety spillway by the construction of a tunnel some eight miles in length through the divide, discharging the surplus waters of the Chagres into the headwaters of the Juan Diaz, or the alternative plan of constructing a tunnel four miles long through the divide, separating the Chagres basin from the headwaters of the Gatuncillo, a stream which enters into the Chagres Valley at Batun. Should this latter course be adopted, it would be necessary to construct an auxiliary channel for the Chagres from Gatun to the sea, in order to divert its flood waters into the bay eastward of Colon.

The third general plan under consideration would be the construction of a canal, with a 20-ft. or 30-ft. level, above the sea, with a single lock at Miraflores and a single lock at Bohio, or in the immediate neighborhood; the construction of the Gamboa dam to be required in this instance the same as in the 60-ft. level plan.

The fourth plan would be the construction of a sea level canal, with a tidal lock, at Miraflores. In this connection it is necessary to explain that, while the mean sea level of the Pacific and the Caribbean is the same, high tide in the Bay of Panama rises 10 ft. above mean sea level and falls 10 ft. below; whereas the fluctuation of the tide of the Caribbean at Colon is less than 2 ft. The construction of a dam at Gamboa, with the necessary spillways, as noted in the previous plan, would be the same under the sea level plan as under the 30 or 60-ft. level.

The construction of the Gamboa dam would provide the water supply for the entire line of the canal, including the cities of Panama and Colon. It would also provide a power plant for the generation of electric power sufficient to furnish ample power for the operation of the Panama Railroad and for the operation of any machinery that might be used in the construction of the canal.

It would require two years to construct this dam, and, roughly estimated, its cost, including spillways, would be between $15,000,000 and $16,000,000, not including the power plant.

Asked by members of the committee for an estimate of the cost of the various plans, Mr. Wallace said the best estimate that could be made at present would be based on the estimate of the former commission of $200,000,000 for a 90-ft. level canal. Figuring with this as a basis, the 60-ft. level canal would cost $225,000,000, could be open for traffic in ten years and fully completed in twelve years; the 30-ft. level would cost $250,000,000, open for traffic in twelve years and fully completed in fifteen years; the sea level canal would cost $300,000,000, could be open for traffic in fifteen years and completed in twenty years.

Chairman Hepburn's questions developed that the estimate of time to build the canal was based on a ten-hour day, and that with the construction of the Gamboa dam and the operation of the power plant, electric light sufficient to illume the whole of Culebra cut might be had, thus making practical the working of two or more shifts, and shortening the time of construction. Mr. Wallace stated that the excavation of this cut was the feature of the construction of the canal that took the time. He said:

On the economical and efficient handling of the material from the Culebra cut depends the cost and the time it will take to complete the canal. Every other question and every other problem connected with the entire work is subordinate and inferior to the problem of the excavation and disposal of the material from the Culebra cut; that is the principal problem of this work. The control of the Chagres, the construction of the various dams, the construction of harbor improvements and all other works are relatively less important.

Work is now going on in the cut, one American steam shovel and some of the French machinery being in operation. Fourteen American steam shovels have been purchased, one of which is being set up. The others are to be delivered at the rate of one a month. In October 3,185 men were on the payrolls of the commission. Of these, 2,165 laborers received 15 cents silver an hour; 245 laborers received 17½ cents an hour; 775 were machinists, boilermakers, pipe fitters, plumbers, carpenters and masons; 256 were Americans from the United States, whose compensation is paid in gold, and who fill positions as engineers, clerks and foremen. More laborers are to be employed in the immediate future. Mr. Wallace is somewhat doubtful of the practical working of the Civil Service order as recently applied to canal employees.

Reverting again to the question of the Bohio dam, on which depends the construction of the 90-ft. level canal, Chairman Hepburn asked questions which brought out a statement from Mr. Wallace as to the probability of finding proper foundation for the dam. Mr. Wallace said:

It is very uncertain. What will be found when more borings are taken is entirely theoretical. But the results of the borings so far taken at the various sites investigated have been disappointing. The average depth bored without finding bedrock has been 163 ft.

As to foundation for the proposed Gamboa dam, which is required in all of the alternate plans, Mr. Wallace said:

At Gamboa the elevation of rock on the site of the proposed dam is at sea level, and numerous borings taken at that point, entering the rock 20 to 25 ft. in depth, would indicate that at Gamboa there is no question as to the ability to find a satisfactory rock foundation for a dam at a maximum depth of sea level.

The probable method of construction, say, of the Gamboa dam, or even of the Bohio dam, would be the construction of a core of concrete and the filling in with the waste dirt out of the Culebra cut. The Gamboa dam can be constructed cheaper than the Bohio dam, on account of the fact that the foundation is much nearer the surface, and the dam site much nearer the Culebra cut.

Answering a question about the operation of a sea level canal, Mr. Wallace said:

A sea level canal would be less expensive to maintain, less expensive to operate, save time in passage through it, and could be widened and deepened, when required, without interfering with traffic.

Mr. Wallace said that he was not prepared to estimate as to the time or cost or plan of a canal until he had completed taking data he is now compiling. He has several parties of engineers at work along the line of the canal, each engaged in compiling data of a given problem, but he could not tell when his recommendations as to the canal will be ready.

RAPID PROGRESS ON CONTRACTS under the Croton Aqueduct Commission has been made during 1904. Based on the monthly estimates the contract value of the work done during the first eleven months of 1904 aggregated $2,384,400, and the probable value of the December work is $315,000. In 1902 the monthly estimates amounted to only $806,400, and in 1901, the highest year from 1895 to 1902, inclusive, the aggregate was $1,282,600. The percentages remaining to be done and the estimated dates of completion for the three principal contracts are as follows: Jerome Park Reservoir (west basin only), 29%; October, 1905. New Croton Dam, 8%; October, 1905. Muscoot Dam, 4%; June, 1905. About one-third of the bottom of the westerly basin of the Jerome Park Reservoir has been completed. Progress here has been unsatisfactory to all concerned. The work is described as the largest "of the kind ever attempted and one for which it was difficult to devise a method for working with great rapidity." About 2,673,000 cu. yds. have been excavated from the easterly basin. The engineer asks for early action on his recommendation that work on this basin be suspended until it is decided whether or not the Croton water is to be filtered, and therefore whether a covering for this basin will be required. The New Croton Dam has been completed to a height of 180 ft. above the foundation and it is expected that by Jan. 15, 1905, the openings through the dam will be closed, so the reservoir can be partly filled this winter. Surveys for additional dams and reservoirs in the Croton drainage area have been in progress since September, 1904. Two sites, one of which was suggested by the Department of Water Supply, have been under consideration and have just been referred to the Commission on Additional Water Supply. It is believed that these reservoirs could be built in two working seasons. The foregoing statements have been taken from a report made to the Aqueduct Commission on Dec. 12 by Mr. J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer.

THE PROPOSED BRONX VALLEY SEWER would cost $1,793,790, according to a report made on Dec. 2 by Mr. Geo. E. Byrne, Chief Engineer to the Bronx Valley Sewer Commission. The sewer would extend from the north line of the town of White Plains, N. Y., to or near the southerly line of Yonkers, and thence along the boundary line between Yonkers and New York City to the Hudson River. The estimates are based on preliminary surveys and allow for a maximum volume of slightly over 200 gallons per head for a total population of 850,000. The upper end of the sewer would be 42 × 63 ins. and the lower end, before changing to tunnel, 64 × 96 ins. The easterly section of the tunnel, from the Bronx Valley to Tibbits' Brook, would be 6 ft. in diameter and the westerly end, which provides for 170,000 more people than the easterly section, 7 ft. in diameter. For the sewer conduit concrete and reinforced concrete, and for lining the tunnel concrete is proposed. Where necessary a paving of either granite blocks or vitrified brick would be used in both the conduit and tunnel sections. The estimated cost of the sewer conduit is $888,200; tunnel, $464,000; coffer-dam and outfall, $22,400; total, $1,374,600; engineering and contingencies (15%), $206,190; land in fee and in easement, $207,000; grand total, $1,793,790. The commission is composed of Messrs. John E. Andrus, Chairman, of Yonkers; Edward F. Brush, of Mount Vernon, and John J. Brown, of White Plains.

THE HUDSON RIVER TUNNEL EXTENSION in New York City, permit for which has been sought by the New York & New Jersey Tunnel Co. of the Rapid Transit Commissioners of the City of New York, has been authorized by that board. It now requires the approval of the Board of Aldermen and of the Mayor in order to become effective. The extension as planned consists of (1) a double-track cross-town subway, running from the tunnel entrance at foot of Morton St. east through Christopher St. and Eighth St. to Second Ave., making a single-track loop at the latter point, and (2) a subway branching from the former and extending up Sixth Ave. to 33d St., where its terminus will be in close proximity to the new terminal of the Pennsylvania R. R. and the Long Island R. R. The river portion of the tunnel, that is the tunnel proper, is already through in the north tube, while the south tube is being rapidly pushed to completion, as recently reported in our columns.

A SUSPENSION BRIDGE FAILED at Charleston, W. Va., on Dec. 15, and two lives were lost in the accident. The bridge crossed the Elk River between East and West Charleston. Its span was about 500 ft. The bridge was very old, having been built before the Civil War, cut down during the war, and later raised again, the old cables being spliced. In recent years it has been considered unsafe for heavy teams. At the time of the failure several school children and two or three vehicles were on the bridge. The cables on one side slipped or broke and the floor in part fell to the river, in part was left hanging on the remaining cables. Two school children were drowned. A more accurate report of the failure will be given in a succeeding issue.

A BOILER ACCIDENT ON THE "MASSACHUSETTS," lying at the League Island Navy Yard, Philadelphia, Pa., killed five men on Dec. 15 and seriously injured several others. A new boiler had been put into the battleship, and it appears that the manhole cover blew off while being tightened under steam pressure. Five men were at work in the room, and three of them were killed outright while two died later.

2,200 QUARTS OF NITRO-GLYCERINE exploded on the Ohio River near Ashland, Ky., on Dec. 17. The explosive was being transported in two large skiffs from the place of its manufacture at Parkersburg to a firm of oil-well contractors at Maysville. Three men who were in charge of the cargo were blown to atoms by the explosion. Log rafts in the vicinity were torn to pieces and a heavy saw log 15 ft. long and 20 ins. in diameter was blown into the air and landed on top of a bluff 200 ft. above the river and over half a mile distant from the site of the explosion. An engineer resident in the vicinity informs us that glass was broken 15 miles away.

THE STEAMBOAT "GLEN ISLAND" BURNED at sea on Dec. 17, with the loss of nine lives. The boat is one which, during the summer, plies between New York City and Glen Island, a pleasure resort in Long Island Sound, and, in the winter, operates on the Sound between New York City and New Haven, Conn. It was while on her night trip to New Haven on the date named that fire started below and about midships, and spread so rapidly that the utmost haste was required to get the crew and few passengers aboard the lifeboats. As it was, nine persons were unable to escape, and were drowned or burned, seven of the nine being members of the crew. An investigation into the causes of the accident will be made by the Federal authorities.

A 50-000-GALLON WOOD WATER TANK at Ansonia, Conn., is reported as having failed on Dec. 13. It is stated that the tank had just been completed by the Consolidated Railroad and that it was being filled for the first time when the bands "burst from top to bottom."

A WOOD WATER TANK AT SAWTELLE, CAL., failed on Dec. 1. Rusted hoops are given as the cause of the failure. The capacity of the tank is reported as 50,000 gallons.

SCARCITY OF WATER SUPPLY IN NEW ENGLAND is reported as seriously affecting users of water power, farmers and others, particularly in New Hampshire and Vermont.

FIRE ENGINE CARS have been added to the equipment of the Metropolitan Elevated Ry., of Chicago, for the purpose of fighting fires that may occur along the line. Within the car are two pressure tanks of 100 gallons capacity for a chemical solution, and also a storage tank of 200 gallons capacity. Each of the smaller tanks has 150 ft. of hose attached. The cars are stationed at the four terminals of the road.

A WATER CURTAIN fire protector for a department store in connection with an automatic sprinkler system has been installed at the new building of the Sweeny Co., at Buffalo, N. Y. A Gould triplex pump driven by a 100-HP. electric motor and with a capacity of 1,000 gallons per minute is piped to a Grinnell sprinkler system, and also to a "water curtain" connection to perforated pipes located along the cornice and over the window frames. The pump was built to the Buffalo Underwriters' Association specification, and is controlled by a combined automatic and hand starting device, by which a constant pressure is maintained in the sprinkler system.

THE LONG CONTESTED PATENT SUIT over the Knibbs patent on automatic relief valves for fire engines, which is said to have been in the courts for over a quarter of a century, has been finally decided by the U. S. Circuit Court of Appeals against the validity of the patent. The inventor, Knibbs, an employee of the Troy, N. Y., fire department, invented in the early '60's a relief valve for fire engines to prevent the hose being overstrained or burst in case the flow of water through it was checked by a wagon passing over it or other cause. The device was put into use in Troy and Hartford and was used for more than two years before Knibbs filed application for a patent on the invention. This delay has now been declared by the court of last resort to be fatal to the validity of the patent. Of course the patent long ago expired and the suit brought was for damages due to infringement of the patent during its life. This fact and the enormous amount of the damages claimed from the city of New York alone, over $26,000,000, were enough to greatly prejudice the case of the plaintiffs in the minds of fair-minded men. The injustice in mulcting the taxpayers of a city for the unauthorized use of a certain small patented device some thirty-five years ago is so manifest to any one that the decision was certainly a most praiseworthy one.

HOT AIR DUCTS for distributing air from a central blower and steam pipe heating system are placed underneath the floor in the plant of the Hope Webbing Co. at Pawtucket, R. I., and thus occupy no valuable space. These underground ducts open into vertical flues which are built into the brick walls. From these flues the air is admitted in the usual manner through openings placed about 8 ft. above the floor level. The entire heating plant was installed by the B. F. Sturtevant Co., of Boston, Mass.

THE INTERSTATE COMMERCE COMMISSION in its annual report just presented to Congress urges anew that it be given the same power to revise rates which it had previous to the Supreme Court decision of 1897. It alludes to the long standing abuses in connection with private cars and to the growing practice of discrimination through the ownership of terminal or switching lines by large shippers. In the matter of safety appliances the Commission renews its appeal for the adoption of the block system, and for the adoption to be made compulsory if the railways will not act without compulsion.

MANUALLY OPERATED BLOCK SIGNALS on American railways are discussed in the report of the Committee on Signaling and Interlocking of the American Railway Engineering and Maintenance-of-Way Association, which has recently been issued. This shows that out of about 165,000 miles of railway covered by the returns made to the committee, 15,937 miles (15¼%) are using the manually-operated telegraph block system, while 942 miles (0.9%) are using the manually-operated and electrically-controlled system. The former system has 7,205 signals and the latter has 1,897 signals.

THE AUSTRIAN CANAL LIFT COMPETITION was decided on Oct. 29, 1904, after half a year of labor on part of the judges. No less than 230 designs had been submitted. A complete report has not yet been published, but it is known that the first two prizes, of 100,000 and 75,000 kr., respectively, were awarded: (1) The Five Bohemian-Moravian Machine Works, in conjunction with the Austrian Siemens-Schuckert Works. (2) The combined design of the Augsburg & Nuremburg Machine Works, the Austrian Siemens-Schuckert Works, the Andritz Machine Works, and Messrs. Offermann, A. Umlauf, L. von Stockert, and W. von Doderer. Three further designs were recommended for purchase and five for honorable mention.

THE WESTINGHOUSE 600-HP. STEAM TURBINE in the Machinery Building at the St. Louis Exhibition was shut down on Dec. 2 after having made a continuous run of over 3,962 hours, having been started on June 20. On examination it was found to be in perfect condition, with no signs of wear, the bearings still showing the tool marks made at the shops. The special feature of this run was the maintenance under load of a speed of 3,600 revolutions per minute, while from 8.30 a. m. to 10.30 p. m. the load varied from 25% underload to 25% overload. The total number of revolutions was 855,792,000.

BREADSTUFFS exported from the United States in 1904 are likely to be less than half the value of the breadstuffs exported in 1903. In the eleven months ending Nov. 30, 1903, the total value of breadstuffs exported was about $176,000,000, while in the corresponding period this year the value exported is only $82,000,000. Nevertheless, the loss of nearly one hundred million dollars in breadstuffs will be about made up in the increase of value of other exports. For the eleven months ending Nov. 30 there was an increase of 10 million dollars in mineral oils, 14 millions in cotton and 64 millions in manufactured goods.